K718

COASTAL ENGINEERING

PROCEEDINGS OF SECOND CONFERENCE

ON

COASTAL ENGINEERING

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PART 1
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REVISED WAVE FORECASTING RELATIONSHIPS

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INTRODUCTION

Data on the generation and decay of wind-generated gravity waves have been collected for several years by the University of California. These data together with the original data by Sverdrup and Munk have been analyzed, and the results were presented in dimensionless graphs suitable for use in wave forecasting (Bretschneider, 1951). No analysis was made of the effect of following or opposing winds.

The dimensionless parameters presented by Sverdrup and Munk (1947) (c/U, gH/U^2, gF/U^2, gt/U, and tU/F) are suitable; however, new curves have been constructed which include the new data recently available (Figure 1). In order that the data on the decay of waves could be presented in a logical manner, a concept, based on the following observations was introduced: (a) Individual waves do not maintain their identity in deep water, (b) a spectrum of lengths and heights is present in both the fetch and decay areas, (c) at any particular decay distance the significant period decreases with time, (d) the significant period increases with decay distance in a manner different than that assumed by Sverdrup and Munk for their decay relationships, (e) the travel time depends upon the group velocity associated with the period at the end of the decay distance.

It is found that the wave height and period at the end of the decay distance depend upon the length of the fetch and the height and period at the end of the fetch as well as upon the decay distance. Using D/F as a dimensionless parameter, decay graphs, D/gT_F^2 versus D/gT_D^2 and D/H_F versus D/H_D, were constructed representing the increase of period and the decrease in height, respectively. These curves give a unique solution for each combination of period and height at the end of the fetch; whereas, the decay relationships proposed by Sverdrup and Munk (1947) gave a unique solution for the increase of wave period regardless of the wave height at the end of the fetch.

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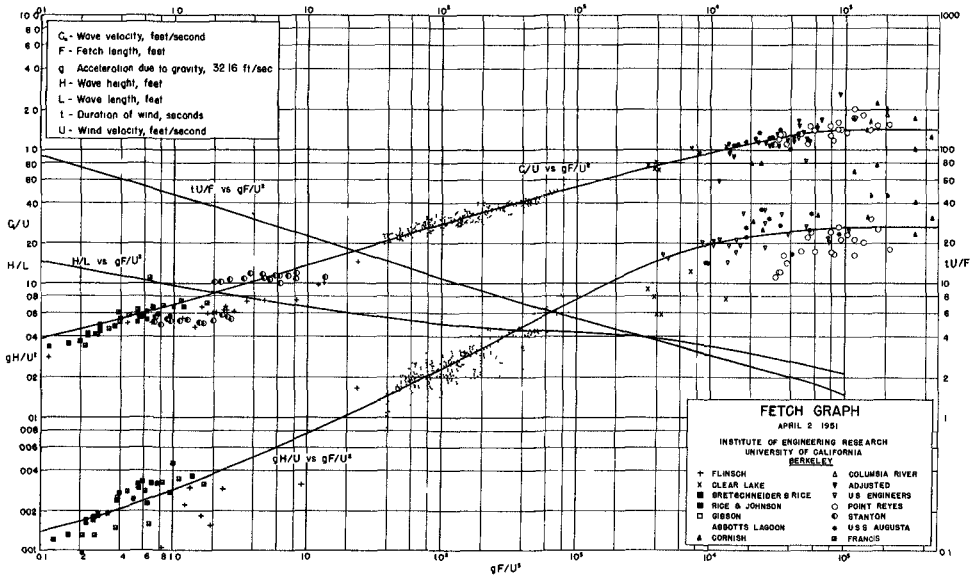
Figure 1   Fetch Graph – Relationship between Wind
Speed, Wind Duration, Fetch, Wave Height,
and Wave Velocity.

Figure 2,   Forcasting Curves for Wave Generation
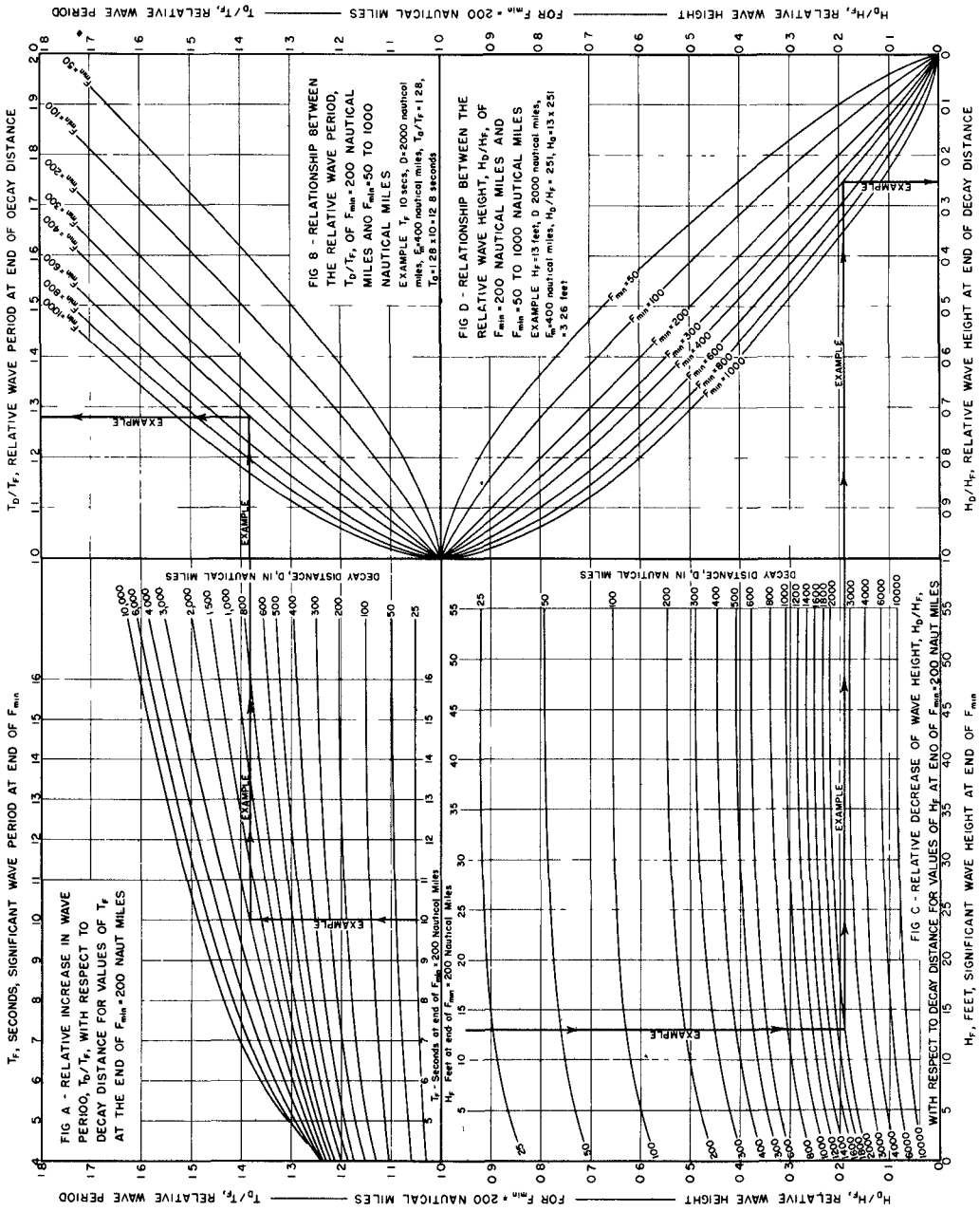
Fig. 3 Forecasting curves for wave decay.

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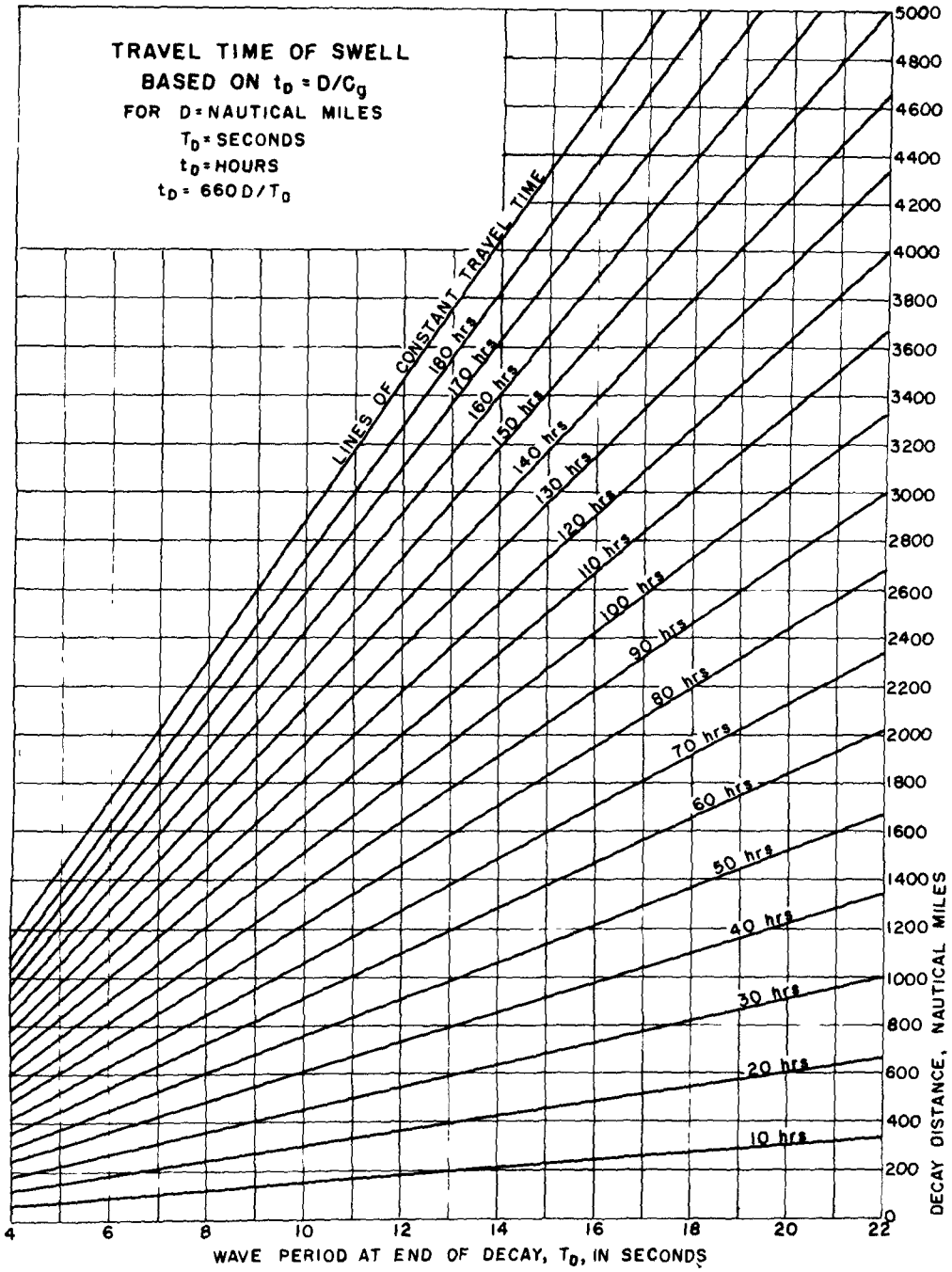
Fig. 4
Travel time of swell - Relationship between travel time,
decay distance and wave period at end of decay.

Forecasting Curves: Figure 2, forecasting curves for the generation of waves, is prepared from Figure 1. Figure 3, forecasting curves for the decay of waves, is prepared from published decay curves (Bretschneider, 1951). Since Figures 2 and 3 are based on much wave data, they may be used to make reliable wave forecasts.

The travel time of the swell is based on the average group velocity of the significant waves at the end of decay and is given in Figure 4.

Example of the use of Figures 2 and 3

Given: U = 23.5 knots (as determined from weather map)

F = 600 nautical miles (as measured from weather map)

t_d = 33 hours (as determined from weather map or maps)

D = 2000 nautical miles (measured from weather maps)

Enter Figure 2 at the left on U = 23.5 knots and proceed until either t_m = 33 hours or F = 600 nautical miles is first reached, and read H_F = 13 feet, T_F = 10 seconds, t_m = 33 hours and F = 400 nautical miles (this is F_{min}*)

Enter Figure 3-A at T_F = 10 seconds, and proceed to decay, D = 2000 nautical miles (this gives T_D/T_F at D = 2000 nautical miles for a significant wave period at the end of fetch (minimum fetch) of 200 nautical miles). Proceed horizontally to Figure 3-B to F = 400 nautical miles and read T_D/T_F = 1.28; T_D = 10 x 1.28 = 12.8 sec.

To determine the wave height, H_D at the end of the decay curves, use Figure 3-C and D, and read H_D/H_F = 0.25; H_D = 3.3 feet.

From Figure 4, the travel time of the swell is t_D = 103 hours.

References

Bretschneider, C.L. (1951). The Generation and Decay of Wind Generated Gravity Waves: Tras. Amer. Geophys. Union (In press)

Sverdrup, H. U. and Munk, W. H. (1947). Wind, sea, and swell; Theory of relation for forecasting: Hydrographic Office, U. S. Navy, Publ. No. 601.

*For a given wind velocity the fetch length behind which a steady state is reached depends only upon the wind duration. For a given duration of wind this steady state fetch depends on the wind velocity and is shorter for weak winds than strong winds. This fetch length is called minimum fetch, F_{min}.

Chapter 2

GENERALIZED WAVE DIFFRACTION DIAGRAMS

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INTRODUCTION

Wave diffraction is the phenomenon in which water waves are propagated into a sheltered region formed by a breakwater or similar barrier which interrupts a portion of a regular wave train (Fig. 1). The principles of diffraction have considerable practical application in connection with the design of breakwaters as discussed by Dunham (1951) at the Long Beach Conference. The phenomenon is analogous to the diffraction of light, sound, and electromagnetic waves. Two general types of diffraction problems usually are encountered: one, the passage of waves around the end of a semi-infinite impermeable breakwater (Putnam and Arthur, 1948), and, second, the passage of waves through a gap in a breakwater (Blue and Johnson, 1949; Carr and Stelzriede, 1951). In general, the theoretical solutions have been found to apply with conservative results, that is, the predicted wave heights in the lee of a breakwater are found to be slightly larger than the height of waves that may be expected under actual conditions. The use of the diffraction theory in breakwater design is made convenient when summarized in the form of diagrams with curves of equal values of diffraction coefficients on a coordinate system in which the origin of the system is at the tip of a single breakwater (Figs. 2a-2b, and 3) or at the center of a gap (Figs. 2c, and 4-6). The diffraction coefficient in this instance is defined as the ratio of the diffracted wave height to the incident wave height and usually is designated by the symbol K'. The procedure in preparing diffraction diagrams appears elsewhere (Johnson, 1950). The purpose of this paper is to present diffraction diagrams to supplement the material of Dunham (1951). For complete details on the application of diffraction diagrams to typical harbor problems the reader is referred to this latter paper.

Semi-infinite breakwater - The generalized diffraction diagram shown in Fig. 3 can be applied to a particular breakwater problem once the characteristics of the design wave have been selected - that is, the height, period and direction of the incident wave from which protection is to be provided. The design wave is selected either from recorded wave data as described by Snodgrass (1951) or by application of the hindcasting procedure outlined by Arthur (1951). As an illustration
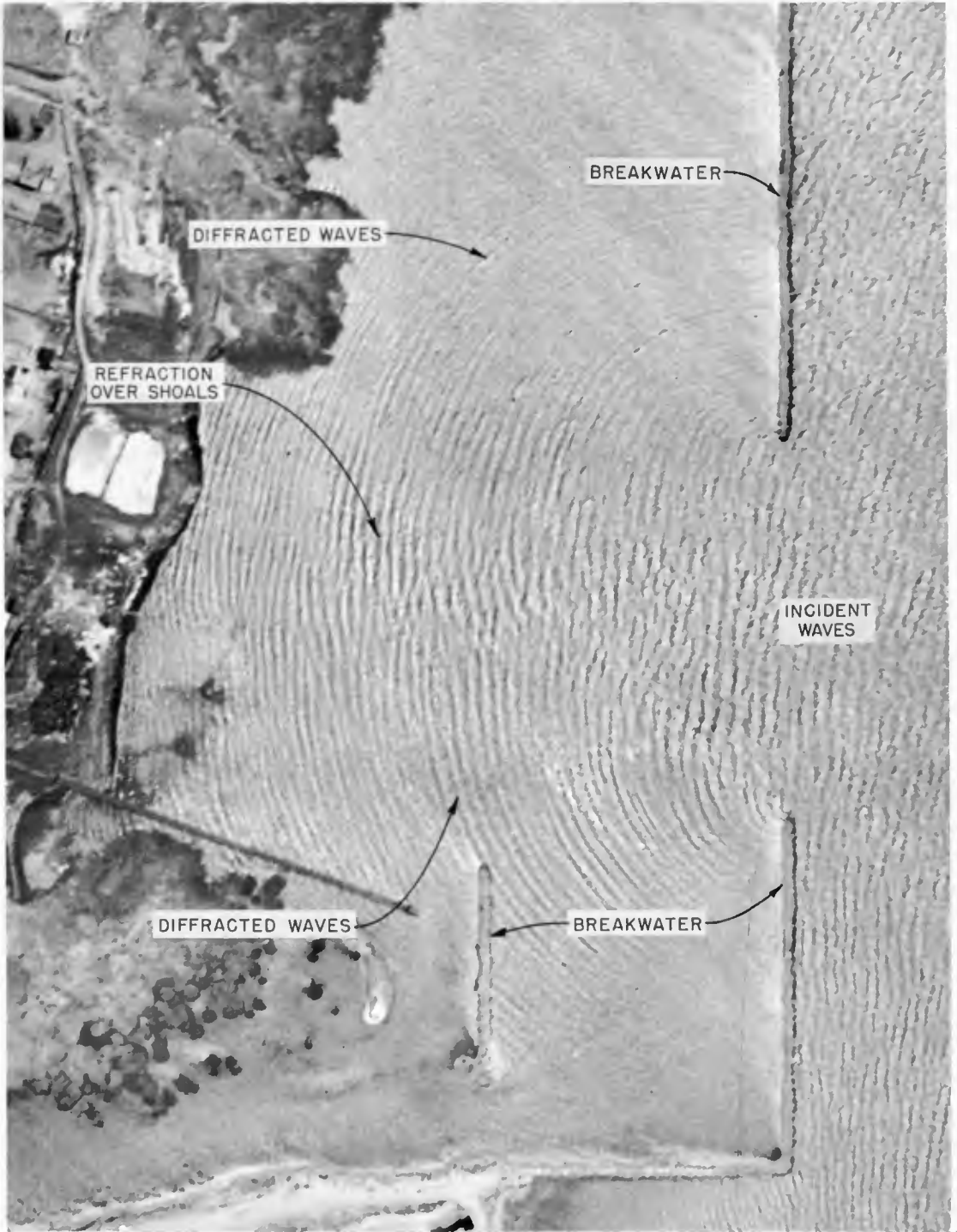
Fig. 1
Diffraction of waves at a breakwater
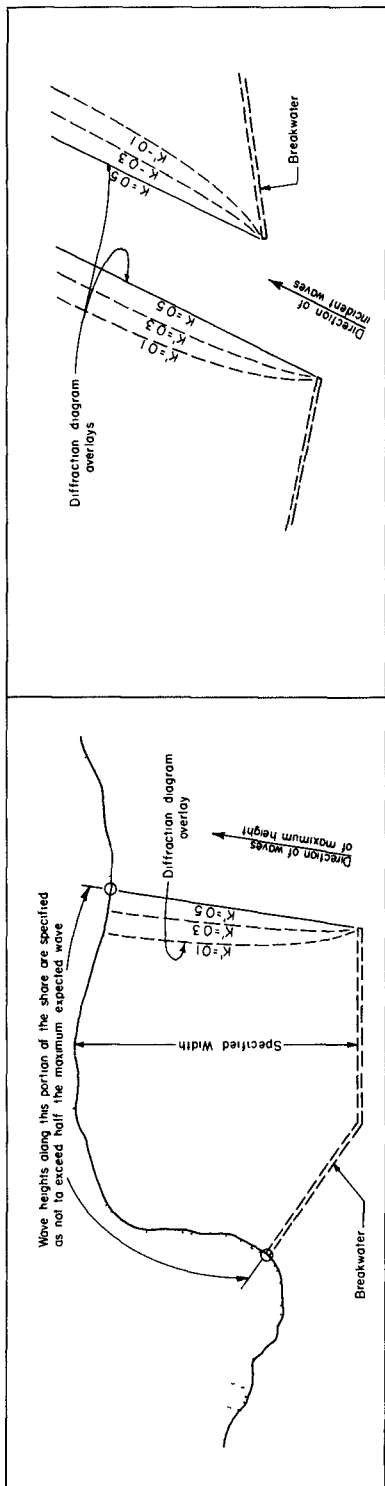
a Semi-infinite breakwater

b Wide gap where diffraction at each tip acts as a semi-infinite
breakwater
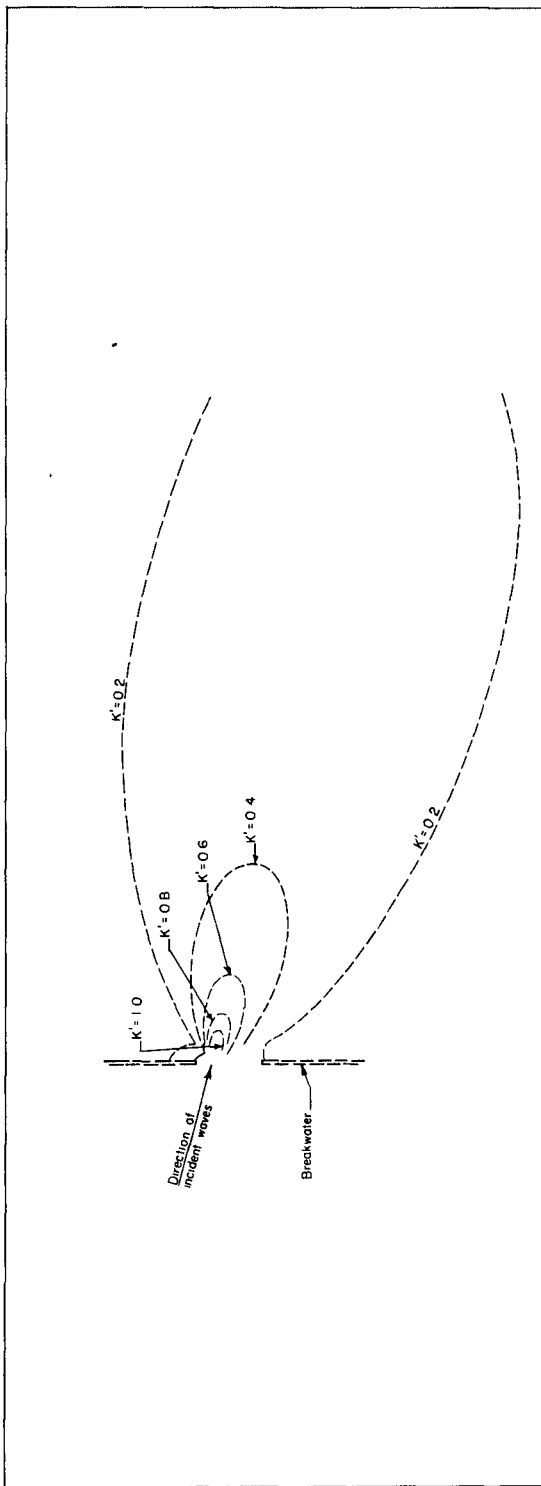
c Breakwater gap

Fig. 2

Typical examples of diffraction at breakwaters

of the use of a diffraction diagram, Fig. 2a shows a map of a harbor
for which protection is desired for a specified reach of the shoreline
with waves approaching from the direction from which the maximum height
waves occur. For a given wave period (or length) a diagram similar to
Fig. 3 is plotted on transparent paper to the same length scale as the
map of the harbor area. This transparent overlay then is moved over the
map (keeping the geometric shadow parallel to the direction of travel)
until the desired degree of protection for the selected reach of the
shoreline is obtained. The location of the tip of the breakwater thus
is obtained as illustrated by the final location of the overlay shown
in Fig. 2a. It should be noted that the diffraction diagram shown in
Fig. 3 is the same diagram as discussed by Dunham (1951) but applied
to both semi-infinite breakwaters and breakwater gaps. The material
summarized below presents diffraction diagrams also for gaps and
permits refinement to the solution of some of the problems discussed
by Dunham (1951).

Diffraction at a Breakwater Gap - The treatment of diffraction problems,
as discussed in the above paragraph is concerned with waves moving past
a breakwater tip with an infinite expanse of water existing away from
the tip. In many harbors, however, waves move through a relatively
narrow gap in a breakwater (Fig. 2c); hence, diffraction occurs at the
two sides of the gap and changes in wave height in the lee of the break-
water will be different than if a single tip existed. Theories for this
condition also have been developed. Experimental studies have verified
the general form of the theoretical expressions for breakwater gap
openings as small as a half wave length. As long as the water depth in
the lee of the structure remains constant the diffraction pattern is
independent of the actual depth. In natural harbors, however, this
condition of uniform depth may not always occur. Instead a shoaling
bottom usually exists - in which case the waves are not only diffracted,
but refraction also results as the waves move further to the lee of the
structure. At a considerable distance from the breakwater, it is probable
that the refraction effects predominate over the diffraction effects.

Figs. 4-6 show generalized diagrams which give diffraction coefficients
to the lee of breakwater gaps of various widths but with normal approach
of the waves. The method of making the necessary computations of these
diffraction coefficients as well as the computations for the position of
the wave fronts (shown only in Fig. 5a) is presented elsewhere (Johnson,
1950; Carr and Stelzriede, 1951). These generalized diagrams, when used
as transparent overlays, can be moved over a map of an area to obtain
the most desirable protection, similar to the procedure illustrated in
Figs. 2a-2b for a single breakwater.

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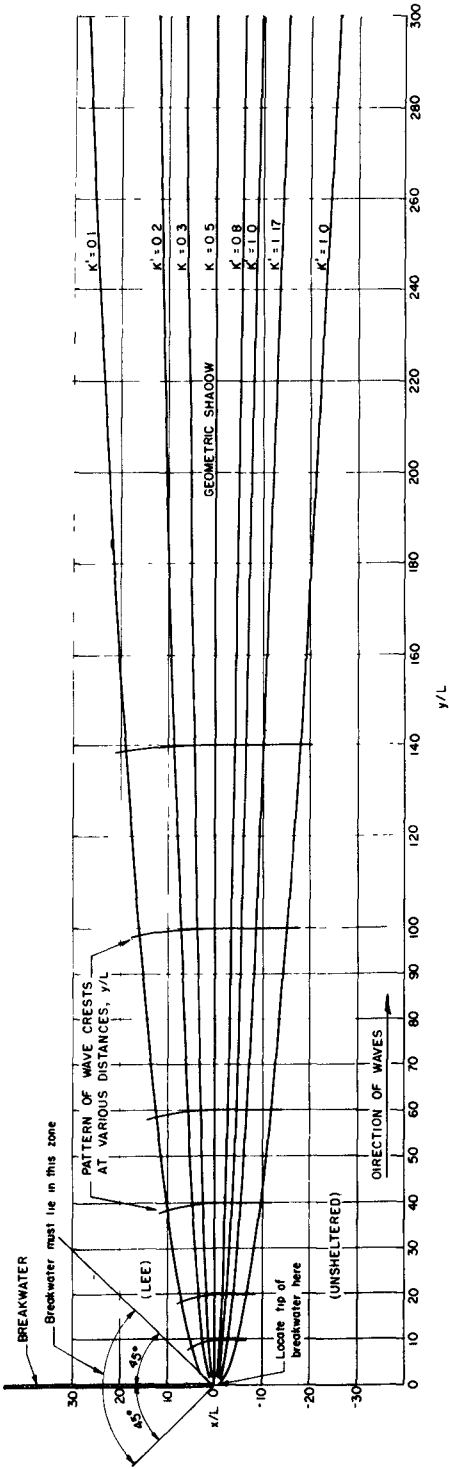
Fig. 3 Generalized diagram for diffraction at the tip of a semi-infinite breakwater
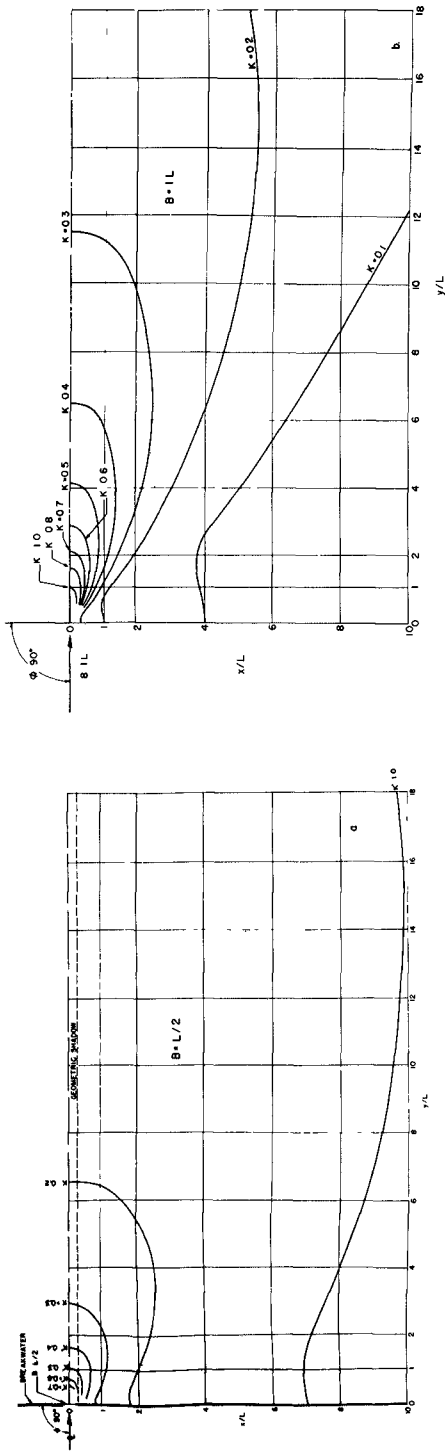
Fig. 4 Generalized diagrams for diffraction at breakwater gaps of one-half and one wave length in width (90 degree approach).

GENERALIZED DIAGRAMS

For semi-infinite breakwaters the single diagram shown in Fig. 3 is sufficient for all such breakwaters with waves approaching from any direction within the limits indicated (also see universal diagram of Dunham, 1951). In the case of breakwater gaps, however, a different diagram is required for each combination of gap width and direction of wave approach. A number of representative generalized diagrams for gaps are shown in Figs. 4-6, inclusive. These diagrams pertain to gaps which range in width from $\frac{1}{2}$ to 5 wave lengths with the direction of wave approach being normal to the gap. These diagrams cover a wide range of gap openings with a sufficiently small spacing of values such that one of the diagrams can be selected and applied with reasonable accuracy to a specific problem. For some specific gap width it may be desirable to obtain the diffraction pattern by interpolation between two diagrams; however, the accuracy with which the design wave data are known usually does not justify such a refinement. In the event though that inter-polation is desired, Figs. 7-10 are provided which show values of diffraction coefficients for various gap widths at various x/L distances from the gap center line and at various y/L distances to the lee of the gap. These curves have been smoothed somewhat to eliminate the unimportant lobes which result from the theoretical solutions as shown in Figs. 4-6. It is to be noted that in Figs. 4-6, inclusive, the diffraction diagrams have been terminated arbitrarily at a distance of 20 wave lengths in the lee of a gap. It is believed that in most applications the effects of refraction, as discussed above, would predominate by the time the waves had traveled a distance of 20 wave lengths beyond a breakwater; there-fore, the extension of the diffraction patterns to greater distances is unnecessary.

When a gap width is in excess of about five wave lengths, the diffraction patterns at each side of the opening are more or less independent of each other (compare Figs. 3 and 6b). In such cases the pattern given by Fig. 3 for a semi-infinite breakwater can be used to estimate the height and direction of waves on the leeward side as discussed by Dunham (1951) and illustrated in Fig. 2b. For these relatively large gap openings the direction of the incident waves with respect to the breakwater alignment can lie anywhere within the zone indicated in Fig. 3 without the diffraction pattern being appreciably affected. For relatively narrow gaps (gap openings of about 3 wave lengths and less) the diffraction pattern can be computed easily by the method of Carr and Stelzriede (1951) for various values of the angle between the incident wave and the breakwater. As an example, Figs. 12-14 show diffraction patterns for waves approaching a break-water gap with a width of one wave length from various directions.
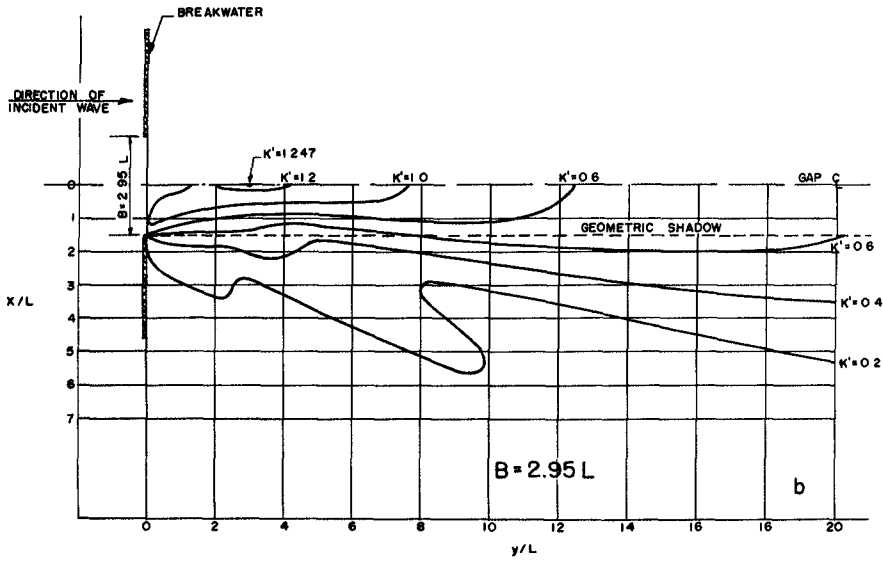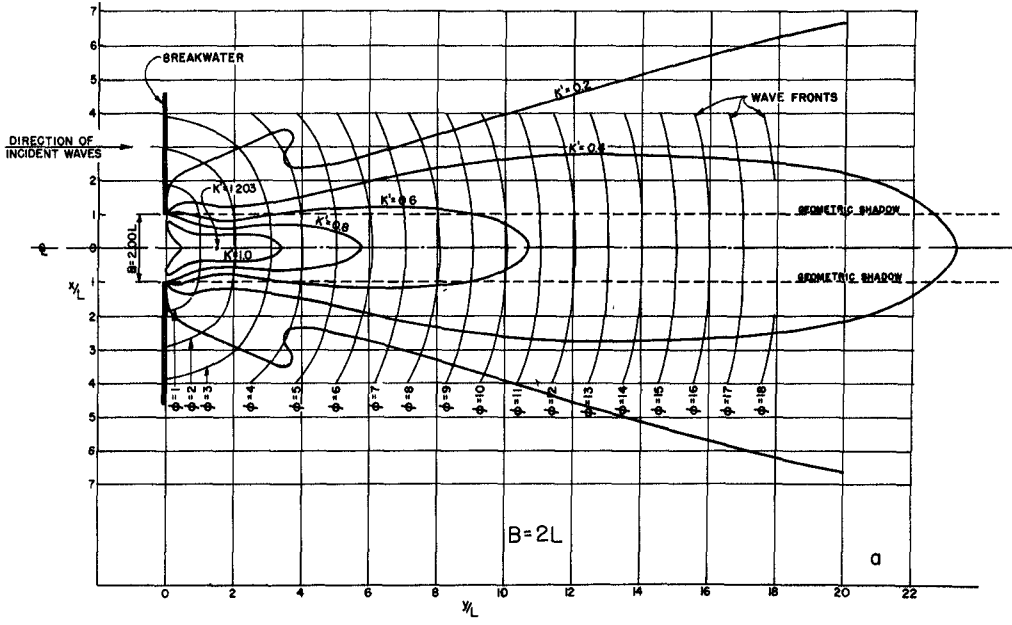
Fig. 5
Generalized diagrams for diffraction at break-
water gaps of two and approximately three wave
lengths in width (90 degree approach).

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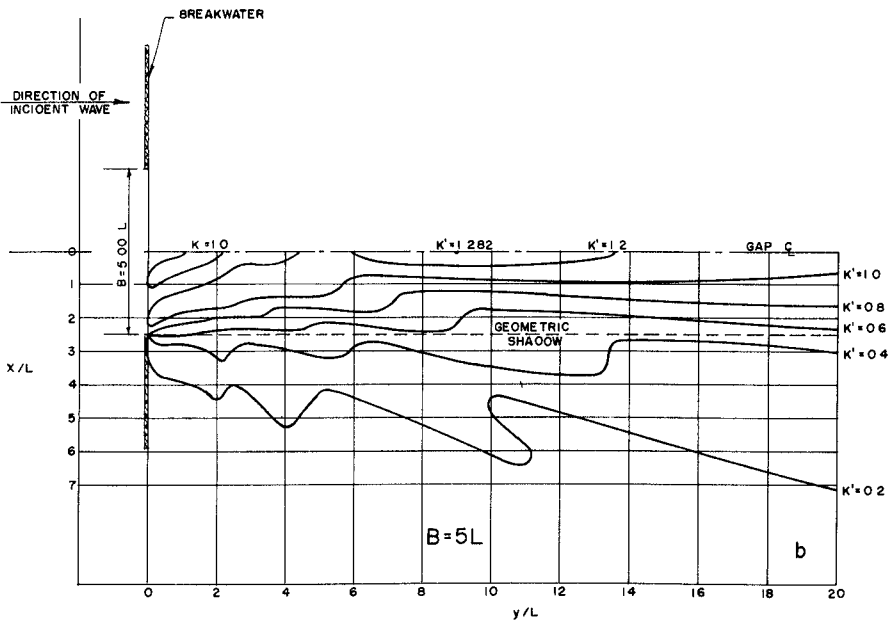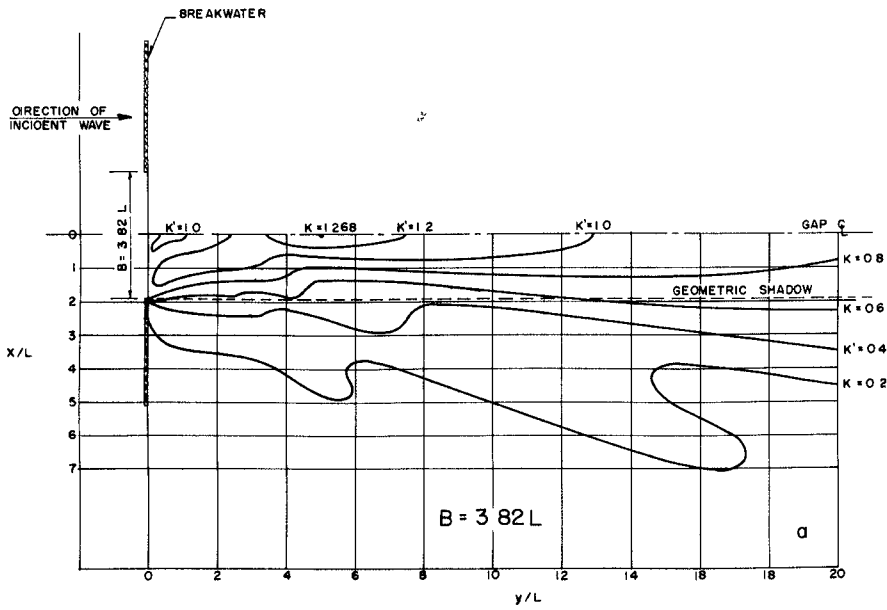
Fig. 6
Generalized diagrams for diffraction at breakwater gaps of five
and approximately four wave lengths in width (90 degree approach).
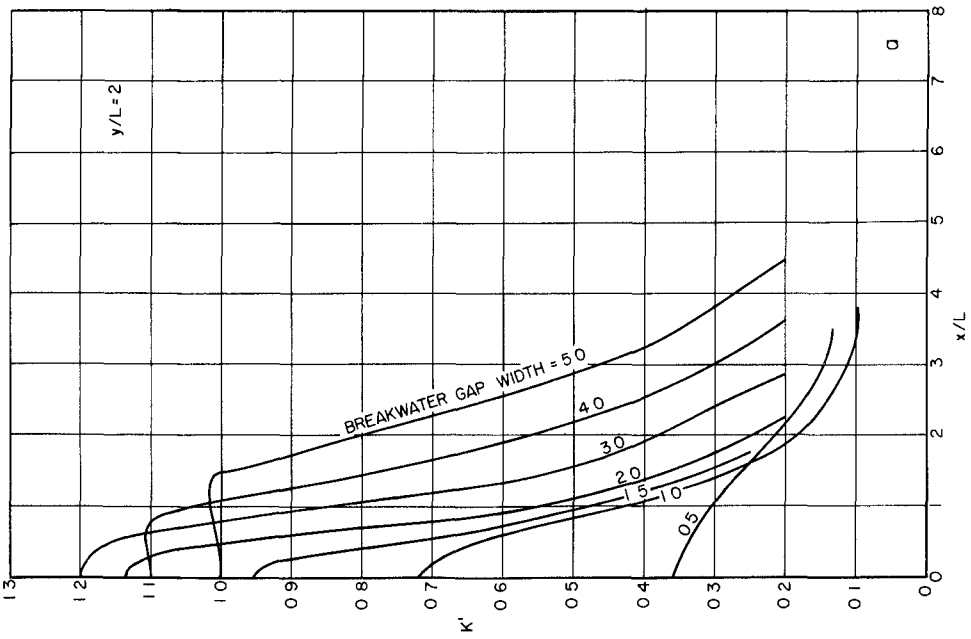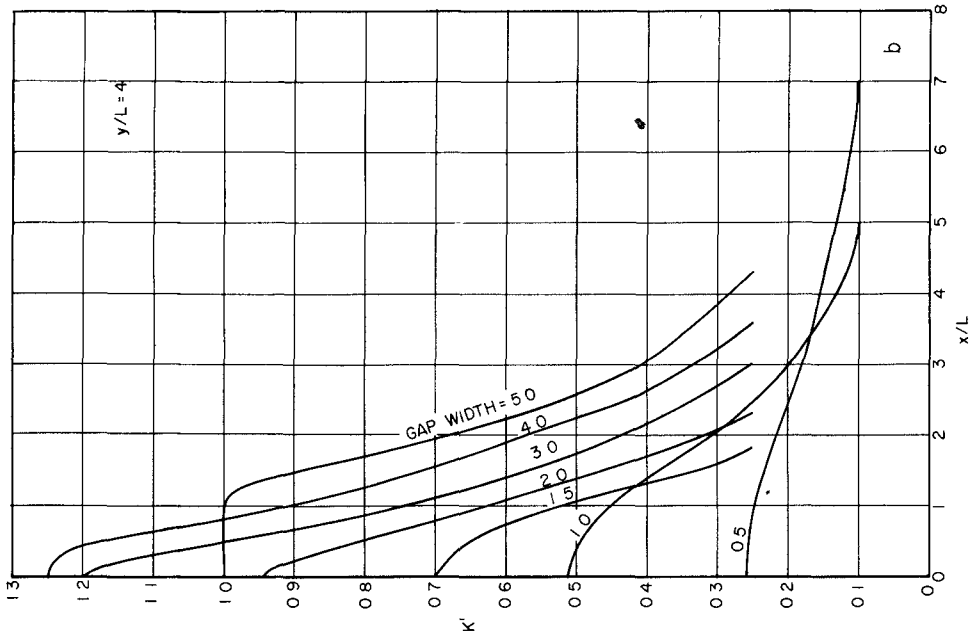
13

Fig. 7 Diffraction coefficients at various points in the lee of breakwater gaps of from one-half to five wave lengths in width (90 degree wave approach).

14
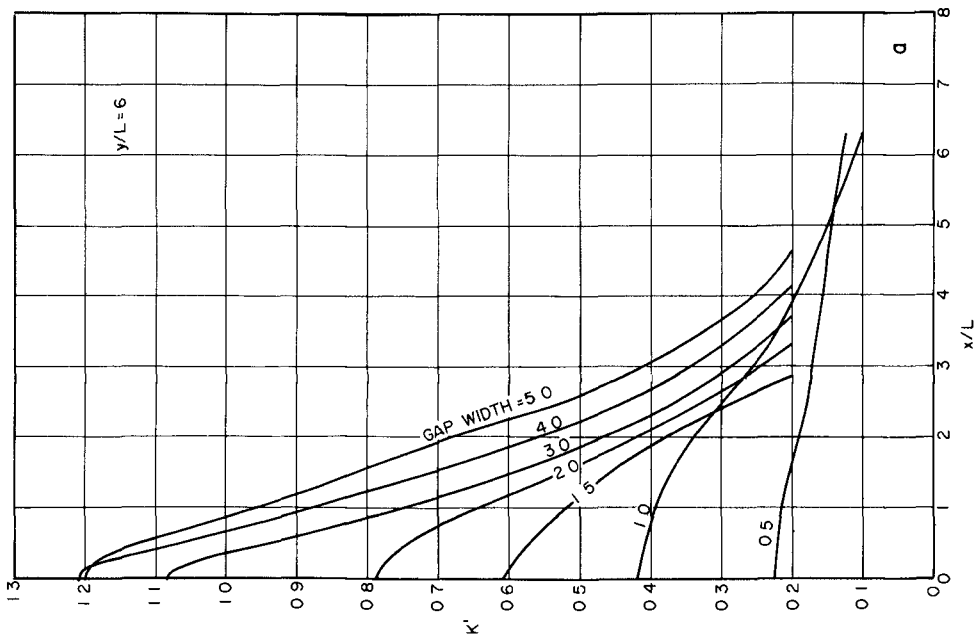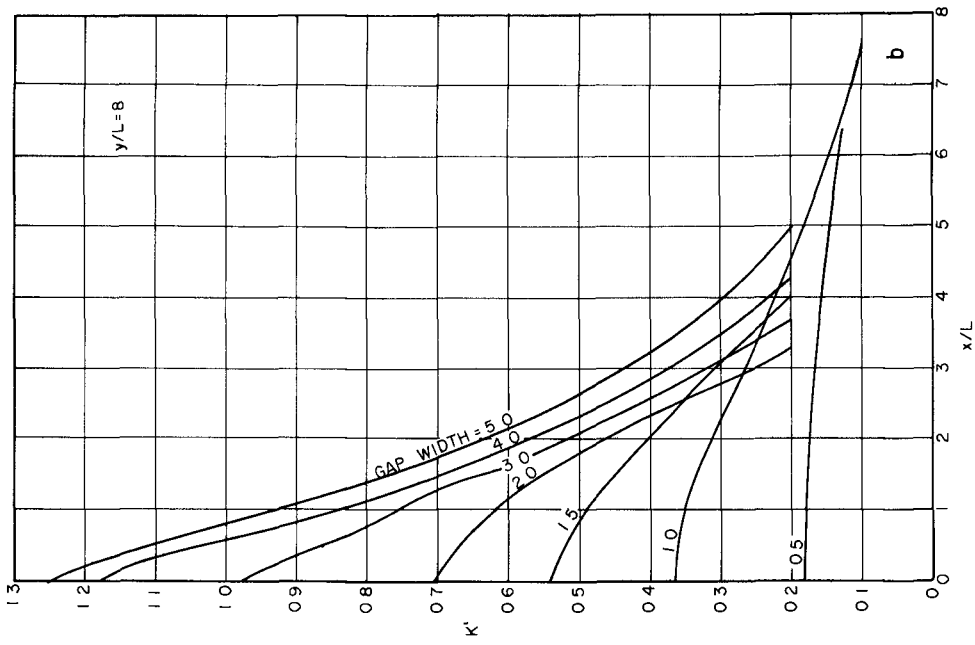
Fig. 8 Diffraction coefficients at various points in the lee of breakwater gaps of from one-half to five wave lengths in width (90 degree wave approach).
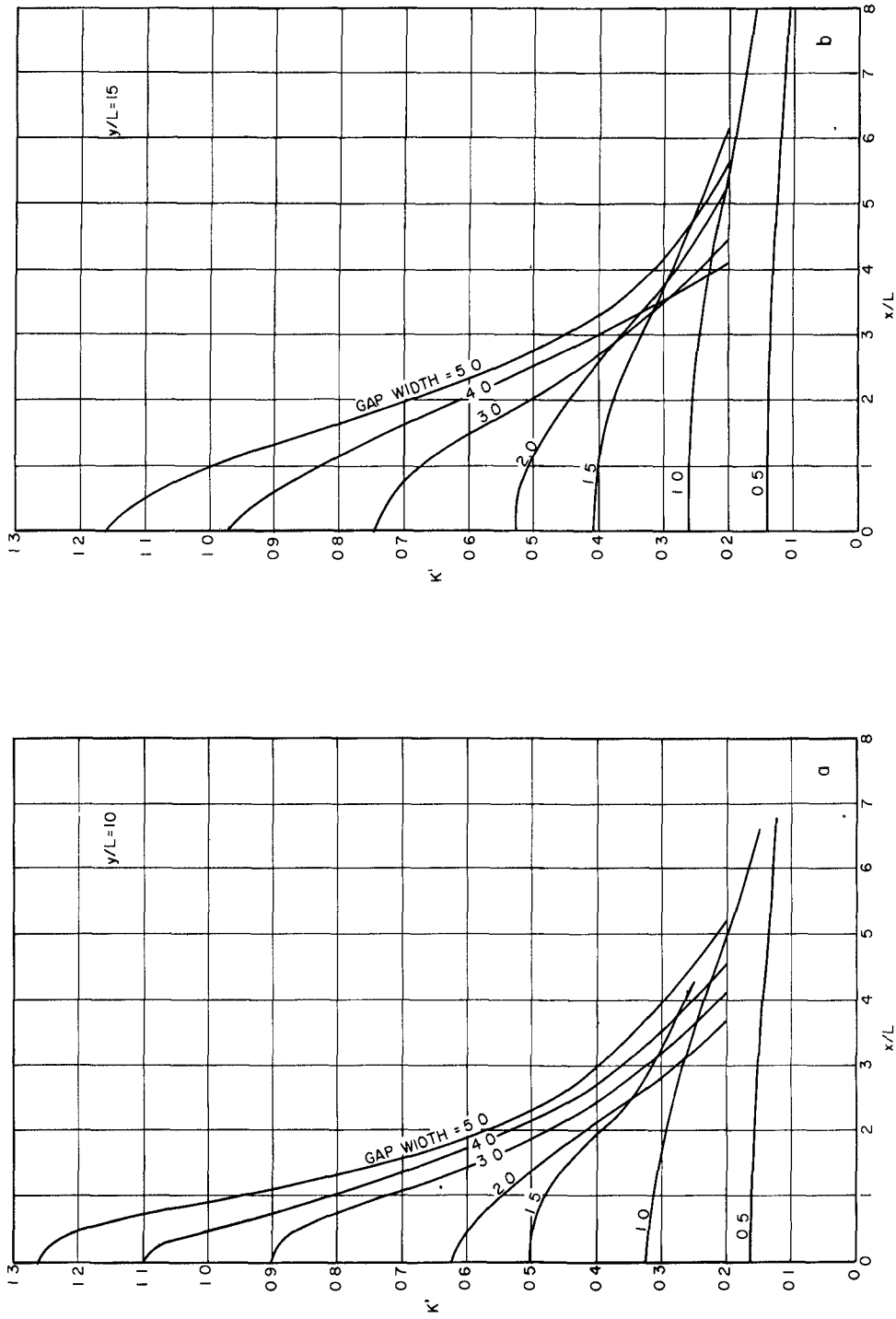
Fig. 9 Diffraction coefficients at various points in the lee of breakwater gaps of from one-
half to five wave lengths in width (90 degree wave approach).
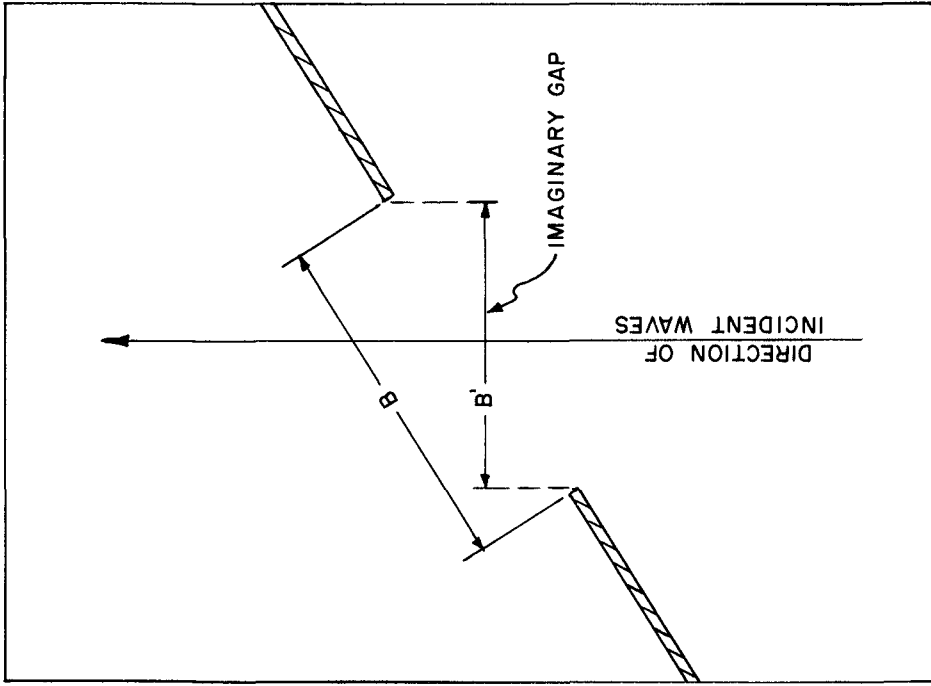
Fig. 11
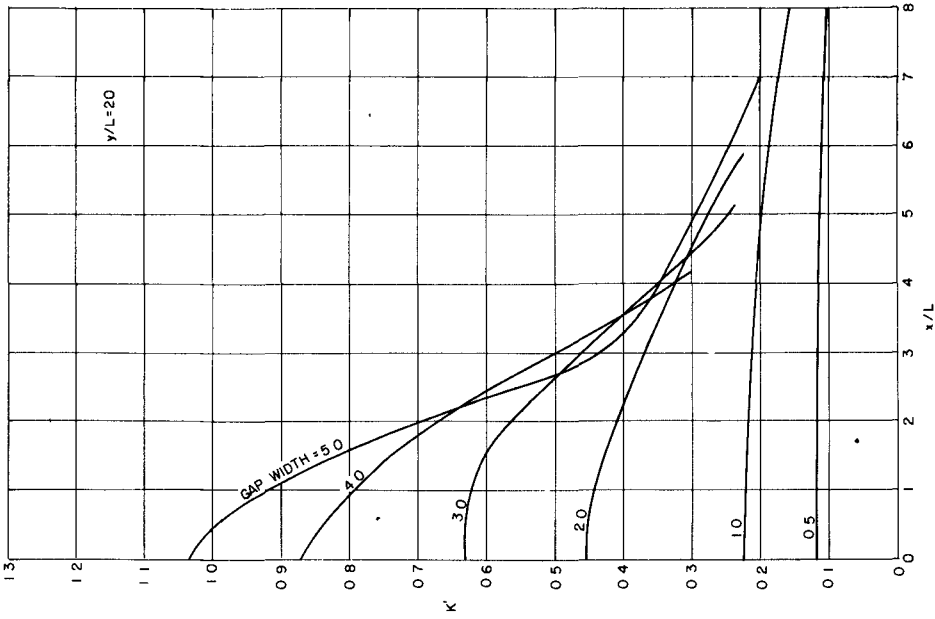
Fig. 10 Diffraction coefficients at various points in the lee of breakwater gaps of from one-half to five wave lengths in width (90 deg. wave approach).
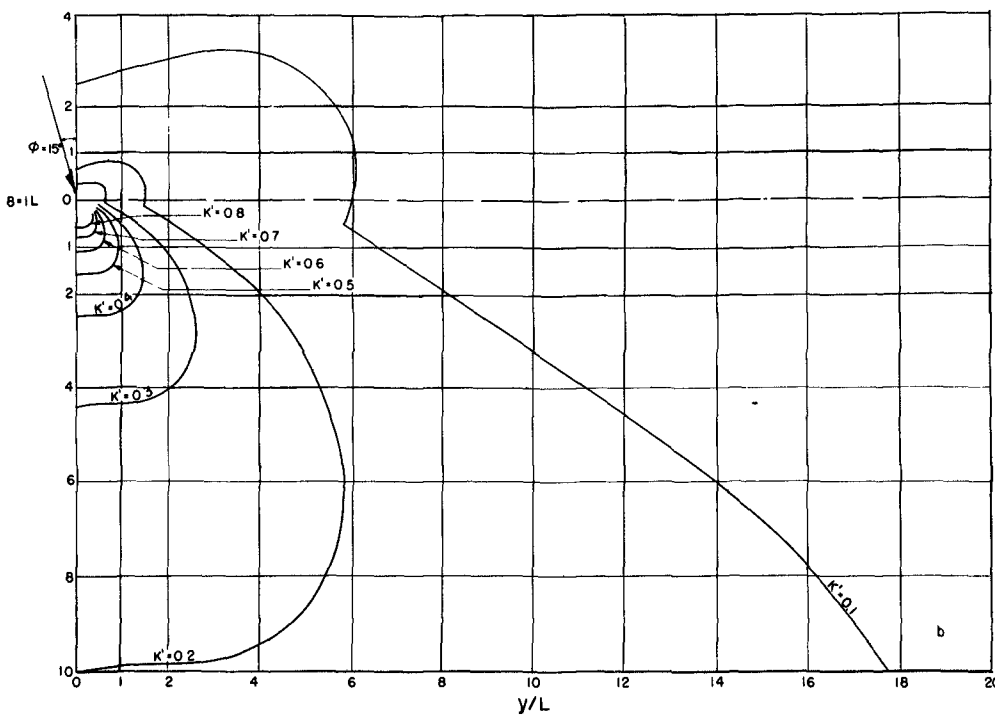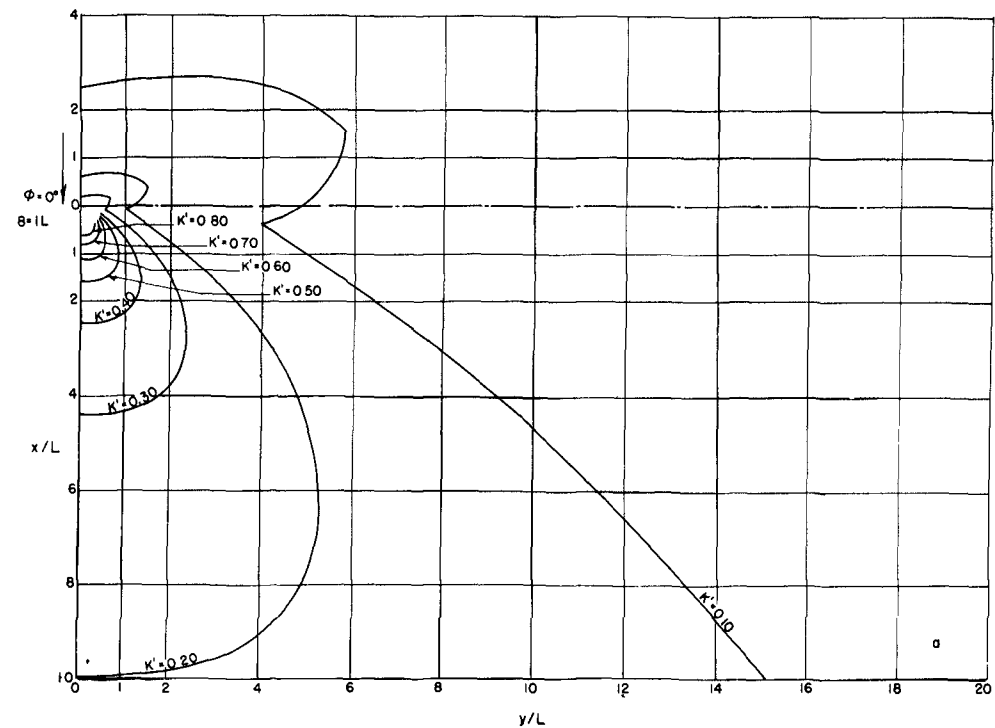
17

Fig. 12
Generalized diagrams for diffraction at
a breakwater gap of one wave length width
(ϕ = 0 and 15 degrees).
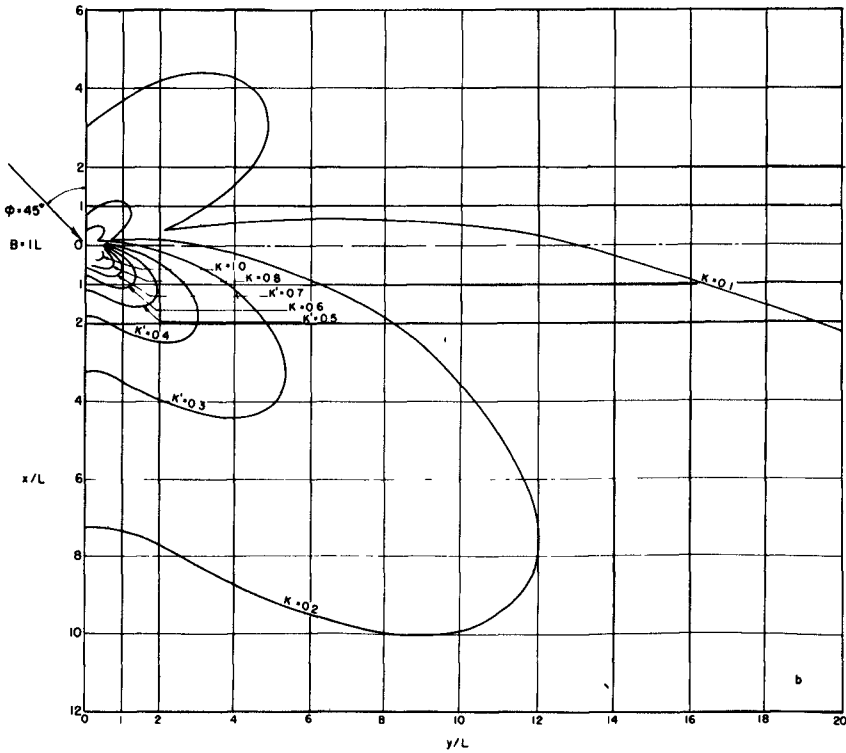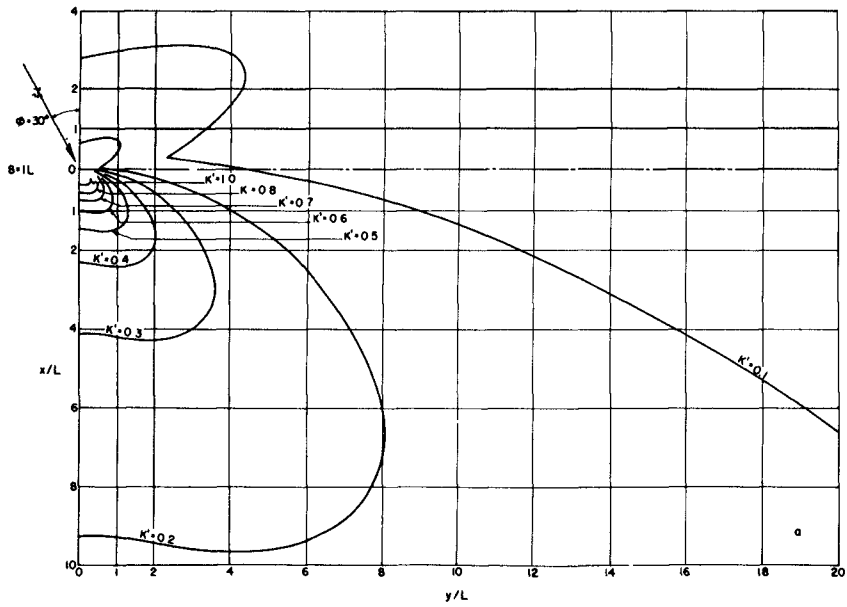
Fig. 13
Generalized diagrams for diffraction at a
breakwater gap of one wave length width
($\emptyset$ = 30 and 45 degrees).

19
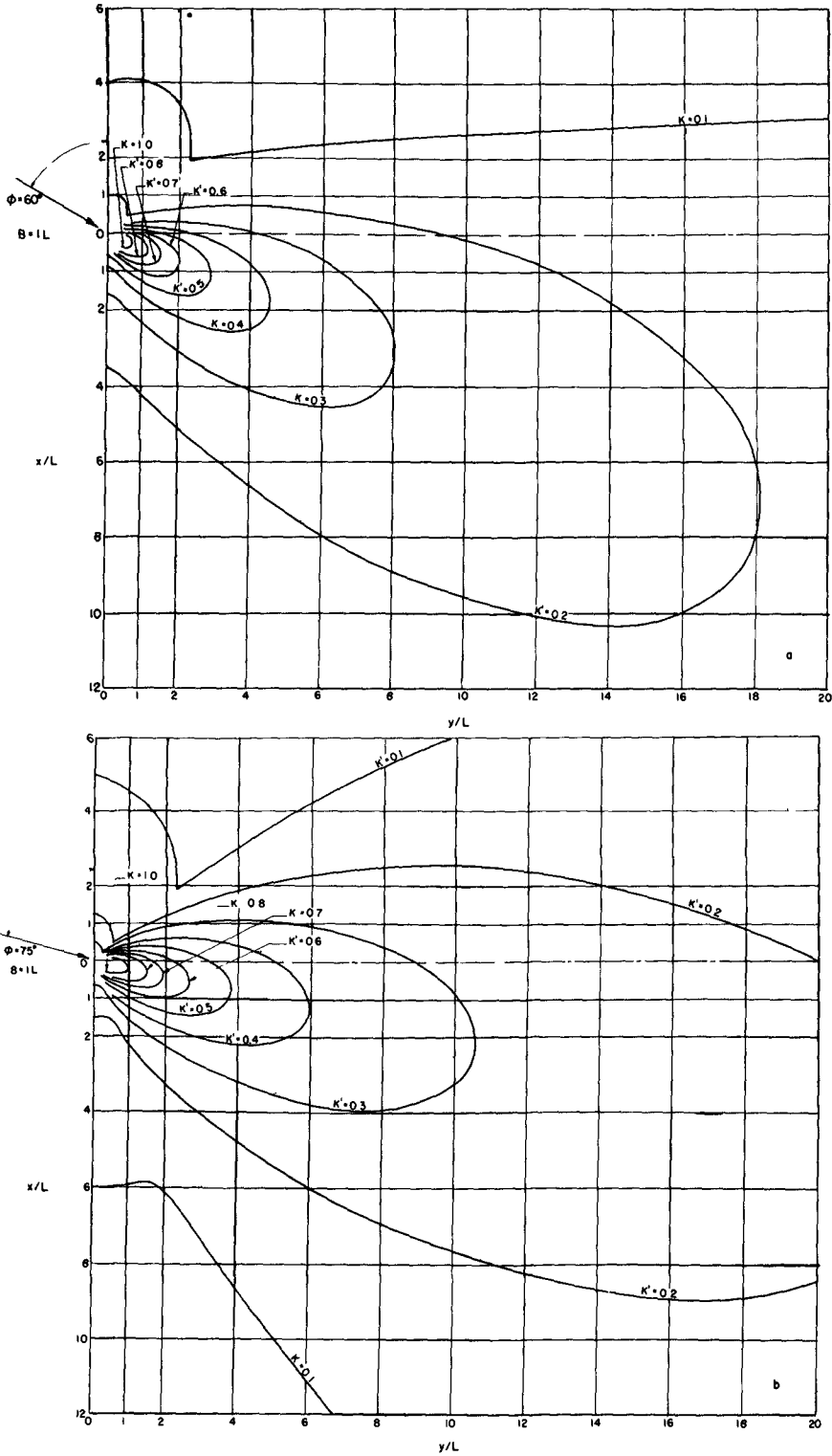
Fig. 14 Generalized diagrams for diffraction at a break-
water gap of one wave length width (∅ = 60 and 75 degrees).
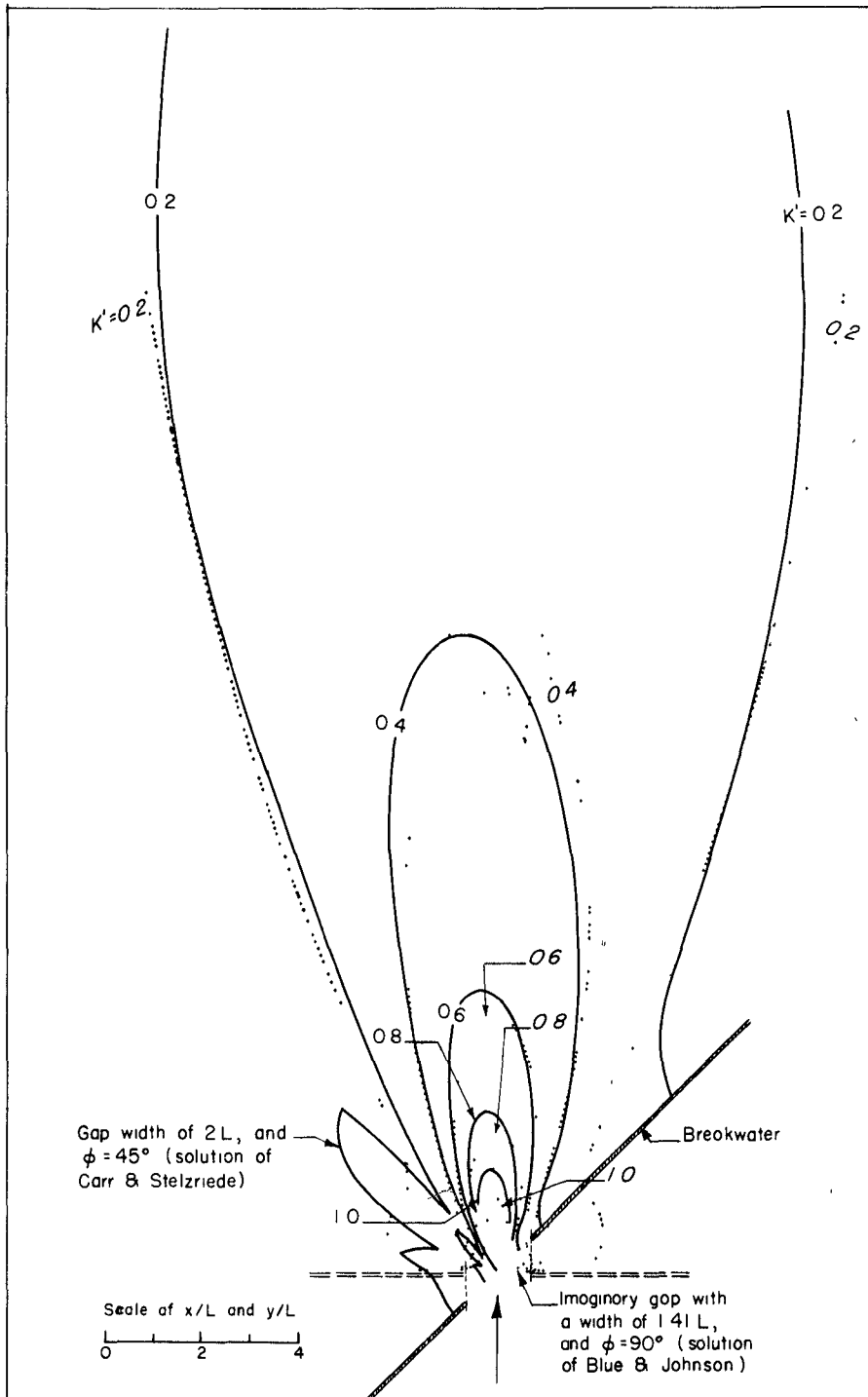
Fig. 15 Generalized diffraction diagram for a gap of two wave lengths and a 45 degree approach compared with that for a gap of width $\sqrt{2}$ wave lengths with a 90 degree approach.

For wider gap openings, where oblique approaches make computations of diffraction patterns relatively difficult, useful approximations can be made by drawling a line through the gap center and normal to the incident wave direction, and then computing diffraction coefficients as though the breakwater were along this line—the end of the imaginary gap being at the projections on this line of the true gap ends (Fig. 11). That this approximation gives acceptable results is demonstrated in Fig. 15 where the diffraction diagrams computed for a gap opening of 2 wave lengths with a wave approach of 45 degrees is compared with a diagram which has been computed for a 90 degree approach to a gap whose width of opening is 1.41 wave lengths. For a given gap opening with an oblique wave approach the width of an imaginary gap for 90 degree approach undoubtedly will be an uneven value. The preparation of a diffraction diagram for such a gap opening is easily accomplished by interpolation from the diagrams shown in Figs. 7-10, inclusive.

SUMMARY

The material summarized in this paper presents generalized diffraction diagrams to be used in the rapid solution of wave diffraction problems which occur in breakwater design. The diagrams are to be used in conjunction with the techniques of application previously described by Dunham (1951).

REFERENCES

Arthur, R.S. (1951). Wave forecasting and hindcasting: Proc. First Conference on Coastal Engineering, Council on Wave Research.

Blue, F.L., Jr. and Johnson, J.W. (1949). Diffraction of water waves passing through a breakwater gap: Trans. Amer. Geophys. Union, vol. 30, pp. 705-718.

Carr, J.H., and Stelzriede, M.E. (1951). Diffraction of water waves by breakwaters: Presented at symposium on gravity waves, National Bureau of Standards, Washington, D.C., June 18-20, 1951.

Dunham, J. H. (1951). Refraction and diffraction diagrams: Proc.
First Conference on Coastal Engineering, Council on Wave Research.

Johnson, J. W. (1950). Engineering aspects of diffraction and refraction: Proceedings Separate No. 122, Am. Soc. Civil Engrs., vol.
77, 1952.

Putnam, J. A. and Arthur, R. S. (1948). Diffraction of water waves by
breakwaters: Trans. Amer. Geophys. Union, vol. 29, pp. 481-490.

Snodgrass, F. E. (1951). Wave recorders: Proc. First Conference on
Coastal Engineering, Council on Wave Research.

Chapter 3

CURRENTS IN THE SURF ZONE[*]

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ABSTRACT

Surface and bottom currents in the surf zone were measured at 15 equally spaced points along two straight beaches with approximately parallel bottom contours. The measurements showed that offshore currents predominate over onshore currents at the bottom, while at the surface there is a slight predominance in the onshore direction. With regard to the longshore component, it was found that surface and bottom currents have a similar velocity distribution. The variability of the longshore component as measured by its standard deviation is equal to or larger than the mean longshore velocity. This wide variation in longshore currents indicates the impracticability of estimating the mean velocity from a single observation of longshore current.

It was found that the momentum approach to the prediction of longshore currents by Putnam, Munk and Traylor (1949) leads to useful forecasts provided the beach friction coefficient k is permitted to vary with the longshore velocity, V. The indicated relation is $k \sim V^{-3/2}$.

INTRODUCTION

A series of longshore current measurements was made in 1949 and 1950 along two straight beaches in the San Diego area. The purpose of the investigation was to study quantitatively the variability of current velocities in the surf zone, and to test the method of prediction of longshore currents from the characteristics of the waves producing them.

The terminology and general principles of the circulation resulting from wave action in and near the surf zone was discussed in a previous paper (Shepard and Inman, 1951). The scope of the present paper is limited to a discussion of currents inside of the breaker zone.

Two straight beaches with relatively parallel bottom contours were selected for study, Torrey Pines beach north of La Jolla and Pacific Beach to the south of La Jolla (Fig. 1). Fifteen stations
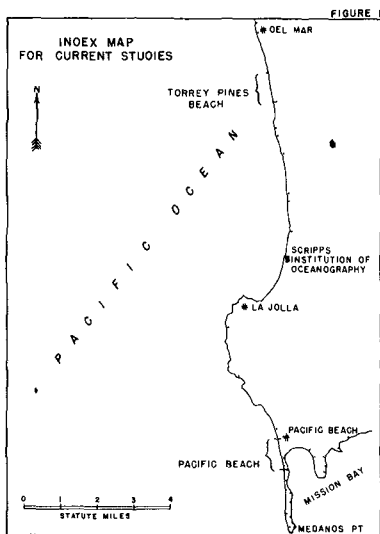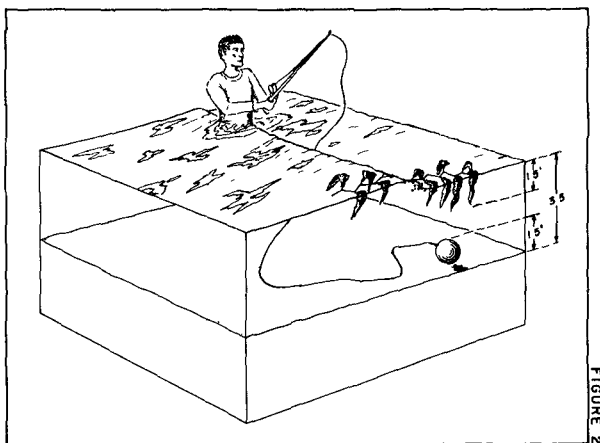
Fig. 1

Fig. 2

Fig. 1. Index map for current studies.

Fig. 2. Devices used in current investigations were of the free drift-
ing type. Kelp was used for surface currents, and a volley ball filled
with water and weighted with sufficient mercury to give it a slight neg-
ative buoyancy was used for bottom currents.
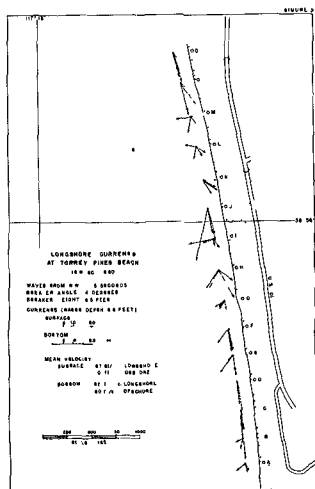
Fig. 3
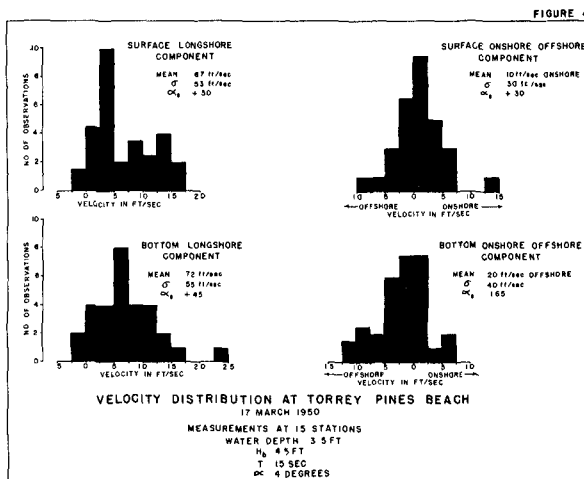
Fig. 4

Fig. 3. A typical series of current observations showing the variation
of currents along Torrey Pines Beach. Each measurement is shown by a
vector opposite the appropriate station.

Fig. 4. Histograms showing the distribution of current velocities for
the series of observations shown in Fig. 3.

were selected at equal intervals along an 0.8 mile stretch on each
beach. The stations were numbered "A" through "O" and were approxi-
mately 300 feet apart. Each series of observations consisted of 2
surface and 2 bottom current measurements at each station giving a
total of 30 surface and 30 bottom current measurements along the
beach. A complete series of 60 measurements required about 2 hours
and were made inside the breaker zone in water approximately $3\frac{1}{2}$ feet
deep.

The devices used in the investigations were of a free drifting
type, the velocity being determined by the distance of travel in a
given period of time. Bottom current magnitude and directions were
obtained by means of a volley ball filled with water and weighted
with sufficient mercury to give it a slight negative buoyancy. The
ball was attached to a light fishing line which was calibrated, and
contained on a reel fixed to a short pole (Fig. 2). An observer,
standing about waist deep at a chosen station in the surf zone,
released the ball at a given signal and played out free line as the
ball was carried by the current. When an observer on shore indicated
that a period of 30 seconds had elapsed, the line was jerked taut and
the direction to the ball noted. The number of feet travelled during
the 30 second interval was observed as the line was reeled in. Direc-
tion of the current was estimated to the nearest $22\frac{1}{2}^{0}$ by noting the
angle that the line made with the shore line. Surface currents were
measured during the same interval by releasing a piece of kelp at
the spot where the bottom current ball was dropped. A second observer
determined the distance and direction of travel of the kelp during the
same time interval. The interval of 30 seconds was chosen as being
long enough to give a representative current and short enough to
indicate major current fluctuation and tendencies in the surf zone.

During current observations, significant breaker heights* were
obtained by an observer standing at mean water level and lining up
the top of the breaker crest with the horizon. The height of the
observer's eye above water level was read from a graduated pole.
This height was multiplied by 4/3 to give the height of the breaking
wave.

The average significant breaker height, H, for the series of
current measurements was obtained by averaging the significant waves
at each station. The significant wave period T, was obtained by tim-
ing the passage of the significant wave crests past a fixed point with
a stop watch. The angle of approach $\propto$, between the crest of the
breaking wave and the straight beach was measured during the 1950
observations by the transit sighting bar method devised by Forrest
(1950). The breaker height H, period T, and the angle of approach
for each series of observations are listed in Table 1.

---

*The significant wave height is the average of the highest one-third
of the waves.

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TABLE I
Tabulation of Field Data

| | | | | | Longshore Component | | | | | | Onshore-Offshore Component [2] | | | | | |
| | | | | | Surface | | | Bottom | | | Surface | | | Bottom | | |
Date	H ft.	T sec.	α[1] deg.	i	V_m	σ	α_3	V_m	σ	α_3	V_m	σ	α_3	V_m	σ	α_3
A. Torrey Pines Beach 1949																
								ft/sec	ft/sec					ft/sec	ft/sec	
1949																
27 Jun.	2.8	15	6½°N	.027				.38	.38	-.75				-.16	.25	+1.36
29 Jun.	3.1	8½	1½°S	.027				.04	.38	-.15				-.06	.28	- .82
30 Jun.	3.7	8	4°S	.027				.22	.43	+.40				-.25	.35	-1.33
1 Jul.	3.6	14	N	.027				.04	.49	+2.50				-.28	.34	+1.50
		9	0°													
5 Jul.	4.9	8	5°S	.027				.84	.51	+1.35				-.05	.51	- .44
6 Jul.	3.8	7	5°S	.027				.21	.45	+ .51				-.33	.36	- .61
7 Jul.	3.4	12½	N	.027				.55	.36	+ .16				-.03	.62	+1.00
		7½	0°													
11 Jul.	2.6	8	N	.035				.04	.40	0				-.38	.21	+1.33
12 Jul.	3.0	9½	1°N	.035				.01	.40	0				-.25	.24	+ .23
13 Jul.	2.7	10	N	.035				.15	.40	0				-.31	.23	- .72
14 Jul.	3.5	13½	N	.035				.09	.45	-1.30				-.32	.48	- .12
		7½	S													
15 Jul.	4.9	13	N	.035				.21	.36	-1.30				-.16	.20	+ .83
19 Jul.	2.9	10	N	.035				.50	.34	0				-.26	.29	+1.78
21 Jul.	4.6	12	N	.035				.80	.66	+ .98				-.16	.39	- .87
26 Jul.	3.7	8	S	.028				.20	.59	0				-.36	-.92	- .21
		15	N													
B. Torrey Pines Beach 1950.																
1950																
14 Feb.	5.1	12	6°S	.027	.37	.84	+ .09	.20	.63	+1.15	-.12	.69	-2.62	-.22	.41	-1.50
		16	5°N	.												
7 Mar.	4.7	14	7°S	.027	.60	.55	+ .12	.45	.60	+1.38	+.32	.78	+ .07	-.21	.39	- .66
17 Mar.	4.5	15	4°S	.027	.67	.53	+ .50	.72	.55	+ .45	+.10	.30	+ .30	-.20	.40	-1.65
29 Mar.	4.8	12	4°S	.027	1.22	1.13	+ .97	1.16	1.05	+ .68	-.11	.16	-2.44	-.28	.45	-1.13
30 Mar.	4.2	12	4½°S	.027	.38	.77	+1.56	.41	.76	- .13	+.01	.41	0	-.27	.68	- .30
		17	3°N													
21 Apr.	2.0	12	4°N	.027	.46	.57	- .49	.26	.46	0	+.07	.47	+3.28	-.09	.23	- .12
11 May	1.7	8	7°N	.027	.10	.50	- .13	.35	.52	- .15	-.01	.35	- .48	-.26	.37	0
		18	3°N													
16 May	2.9	15	5°N	.027	.60	.51	+ .13	.52	.40	- .75	+.08	.25	- .97	-.35	.40	+ .84
25 May	1.6	6	5°S	.027	.11	.33	+1.00	.10	.37	+ .82	-.08	.13	+1.62	-.38	.23	- .07
		14	2½°N													
C. Pacific Beach 1950																
1950																
24 Feb.	6.2	14	5°S	.014				.54	.48	·0				-.35	.53	- .85
2 Mar.	3.1	16	7°N	.014	.82	.47	- .10	.41	.28	+ .49	.03	.66	- .21	-.17	.24	-1.33
14 Mar.	4.5	12	3°S	.014	.46	.94	- .10	.51	.80	- .30	.27	.84	- .20	+.17	.71	+ .63
22 Mar.	3.5	14	4°S	.014	.20	.61	- .92	.13	.54	0	.07	.45	+1.78	-.32	.60	+1.22
18 Apr.	2.7	16	3½°N	.014	.01	1.20	- .07	.25	.70	+ .35	.26	1.14	-1.63	-.21	.68	- .33
9 May	4.7	13	7°S	.014	1.54	1.40	- .48	1.20	1.15	+ .07	.22	.82	+ .34	-.57	1.07	+2.91
18 May	2.6	11½	2°S	.014	.06	.82	+1.72	.01	.64	+1.00	.01	.57	+1.06	-.71	.41	-1.39
		8	N													
26 May	2.0	14½	4°N	.014	.03	.54	-1.78	.19	.42	+ .10	-.17	.32	-1.17	-.54	.37	+1.83
27 July	1.8	12	2½°N	.014	.09	.72	- .31	.03	.49	+ .38	.16	.50	- .43	-.40	.39	+ .43

[1]/The letters N (north) and S (south) indicate the direction of opening of the angle between the breaking wave and the beach. The period and breaker angle of minor wave trains, when present, are listed below the major train.
[2]/Positive values of V_m indicate onshore directions and negative values offshore directions.

STATISTICAL ANALYSIS OF OBSERVATIONS

A typical series of measurements is shown graphically in Fig. 3, in which the velocity and direction of each current observation is represented by a vector opposite the appropriate station. In order to treat the distribution of currents systematically, each current vector was resolved into a longshore component and an onshore-offshore component. The range of velocities in each component was divided into equal classes, and the histogram of the distribution of velocities was obtained by plotting the classes in feet per second as the abscissa and the number of observations in each class as ordinate. Histograms, showing the velocity distribution in a longshore direction and an on-shore-offshore direction for both surface and bottom currents for the series of measurements shown in Fig. 3 are given in Fig. 4.

Since histograms do not give numerical descriptions of the current distribution, the data were treated statistically to obtain the arith-metic mean velocity V_m, the standard deviation σ, and the skewness α_3 of the velocity distribution.

These measures are described in statistics textbooks (for example Hoel, 1947, pp. 8-15) and defined as follows:*

$$V_m = \frac{1}{N} \sum_{i=1}^{h} V_i f_i \tag{1}$$

$$\sigma^2 = \frac{1}{N-1} \sum_{i=1}^{h} (V_i - V_m)^2 f_i \tag{2}$$

$$\alpha_3 = \frac{1}{N\sigma^3} \sum_{i=1}^{h} (V_i - V_m)^3 f_i \tag{3}$$

where N is the number of observations ($N = 30$ for most series), V_i is the class mark (value of the mid-point of the class interval) of the ith class, f_i is the number of observations in the class, and h is the number of classes.

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*$N-1$ was used in the denominator of equation (2) to give an unbiased estimate of the standard deviation in accordance with small sample theory (Hoel, 1947, p. 129).

The standard deviation is a measure of the spread of the distribution, and for most symmetrical distributions approximately 68% of the observations are included between the values $V_m - \sigma$ and $V_m + \sigma$. The skewness, α_3, serves as a measure of the symmetry of the distribution. An α_3 value of 0 is indicative of a symmetrical distribution, while a positive value indicates the distribution is skewed to the right, and a negative value that it is skewed towards smaller or negative values. For purpose of comparison, the mean, standard deviation, and skewness are listed opposite each of the histograms in Fig. 4. These three statistical measures were computed for each component of velocity of each series of observations and are listed in Table I.

DISTRIBUTION OF CURRENTS

Fig. 4 illustrates some of the features that are typical of the current distribution in the surf zone. For example, the surface and bottom longshore components have similar mean velocities, but the distribution of velocities tends to be more symmetrical for the bottom component. This may be because the effect of wind is less near the bottom. The negative current values for the surface and bottom longshore components indicate that a small percentage of currents were moving in a direction opposite to that of the predominant current. The bottom onshore-offshore currents had a pronounced offshore tendency, whereas the mean velocity of the surface currents was onshore.

The nature of the distributions of the bottom longshore current component for all series of observations are summarized in Fig. 5. In this figure the minimum observed velocity, the standard deviation of the velocity distribution, the maximum velocity, and the skewness of the distributions are plotted against the mean bottom longshore velocity. This diagram does not consider such important factors as wave height, period, and angle of approach, and although the plotted points show considerable scatter, they nevertheless illustrate several important features of the bottom longshore current:

(a) In almost all cases, there were currents opposed in direction to the dominant current.

(b) The variability of the current as measured by the standard deviation averaged $\frac{1}{2}$ foot per second for mean velocities below approximately $\frac{1}{2}$ foot per second and was approximately equal to the mean velocity for velocities above $\frac{1}{2}$ foot per second.

(c) The maximum observed velocity increased with increasing mean velocity; the maximum being approximately five times greater than the mean for mean velocities near 0.2 feet per second, and three times greater for mean velocities near 1 foot per second.

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(d) There is no apparent relationship between the skewness of the velocity distribution and the mean velocity. In general the velocity distribution tends to be fairly symmetrical over the range of mean velocities investigated.

The relation of the means and of the standard deviations of the surface and bottom longshore currents are given in Fig. 6. While a linear relationship exists between these quantities, in general the mean value of the bottom longshore current appears to be more consistant in its agreement with the wave conditions generating the currents than the surface velocity, and the spread or variability (as measured by the standard deviation) was somewhat less for the bottom currents than for the surface currents. For these reasons the mean bottom longshore current was used for comparing the observed with the predicted currents in the following section.

The large variation in the observed longshore currents are caused in part by the variation in wave height with time, and by the variation of the cell-like circulation pattern of the nearshore current system with distance along the beach. Since the mass transport of water is proportional to the square of the wave height, groups of high waves followed by groups of low waves result in fluctuations of current velocity with time. (Shepard & Inman, 1950, Fig. 11). Also, in many instances, a secondary wave train was present which may have contributed to the variability.

The nearshore circulation has been shown to have a cell-like pattern, consisting of relatively wide stretches along the beach where water is transported shoreward by the waves, then along the shore inside of the breakers (by longshore currents) into relatively narrow zones where the water is transported seaward by rip currents (Shepard and Inman, 1950, Figs. 2 and 3). This circulation pattern results in higher velocities up current; and lower or in some cases a reversal of longshore current direction down current from the rip zones. This effect is shown in Fig. 3 at stations G and L. Also, high waves approaching with crests parallel to the beach often result in strong longshore currents for limited distances between rip zones, although the mean longshore current for the entire beach may be quite low.

For the above reasons it is advisable to measure currents at as many different stations as possible, in order to obtain measurements that are representative of the beach as a whole. The bias exhibited at the 15 stations on Torrey Pines Beach for the 1950 series of observations is shown in Fig. 7. The plots on this figure show the degree of the divergence between the average current at each station and the mean of all currents for the entire beach. For example, on the average, the mean bottom longshore component at Station M is 60% less than the mean for the entire beach, while the rip tendency as measured by the bottom offshore component is 150% greater.

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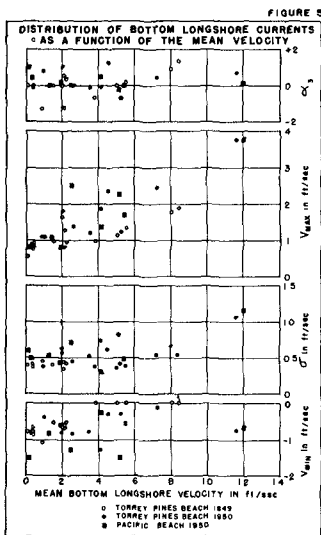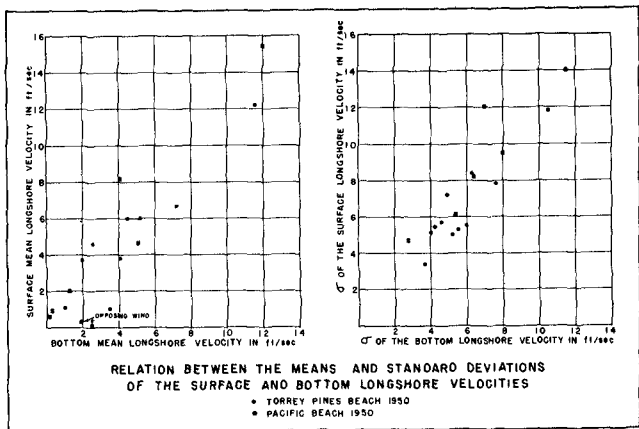
Fig. 5

Fig. 6

Fig. 5. Summary of the distribution of bottom longshore current components as a function of the mean velocity. The negative values for the minimum observed velocity, V_{MIN}, indicate that these currents were flowing in an opposite direction to the mean current. The lower left hand diagram in Fig. 4 shows the histogram of the velocity distribution for one of the plots in this figure.

Fig. 6. Relation of the means and standard deviations of the surface and bottom longshore current components. Compare the upper and lower right-hand histograms of Fig. 5.
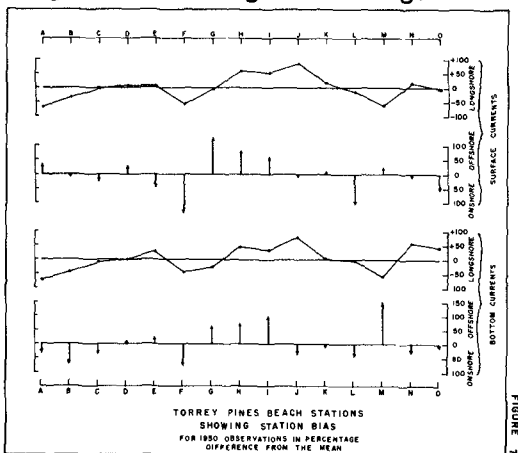
Fig. 7
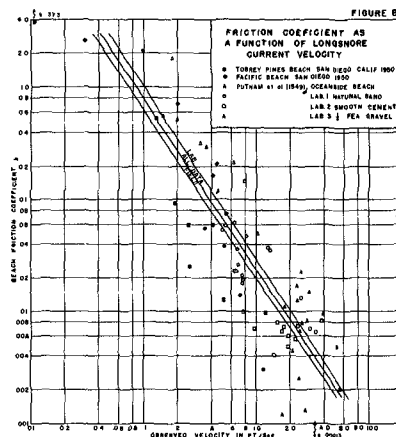
Fig. 8

Fig. 7. The bias exhibited at the 15 stations along Torrey Pines Beach. The plots show the magnitude of the deviation of the average current at each station for the mean of all currents for the entire beach during the 1950 series.

Fig. 8. Relation between the beach friction coefficient k and the observed longshore velocity. The mathematical relation is given by equations (6), (7) and (8). For the Torrey Pines and Pacific Beach observations, the mean bottom longshore component of velocity V_m, listed in Tables 1B and 1C was used.

The variation in currents as represented by the standard deviation of the distribution, σ, is a measure of the accuracy of prediction of the mean bottom longshore current from an individual measurement of current. Obviously, the greater the standard deviation or spread of velocities, the less is the probability of accurately estimating the mean from a single observation of current. For example, Fig. 5 shows that mean bottom longshore currents of $\frac{1}{2}$ foot per second are characterized by a standard deviation of approximately $\frac{1}{2}$ foot per second; this means that out of 100 estimates based on single observations of current 68 of the estimates should fall within 1 standard deviation either side of the mean, or within the velocity range of zero to 1.0 foot per second. The coefficient of variation, $C_v = 100 \ \sigma / V_m$, is a useful measure in this respect, because it gives the deviation from the mean in terms of the percentage of the mean. For the example above, $C_v = 100\%$, and thus 68 of the estimates should fall within plus or minus 100% of the true mean velocity.

The coefficient of variation was computed for all bottom longshore current distributions with mean values greater than 0.1 feet per second.[*] The average of all of the coefficients of variation in this case was 177%. Inspection of Fig. 5 shows that for low velocities, C_v decreases with increasing mean velocity, but tends to be constant for mean velocities of $\frac{1}{2}$ foot per second and higher. The average value of C_v was 91% for mean velocities above $\frac{1}{2}$ foot per second.

It is apparent that if the estimation of the mean velocity is based on more than one observation, the accuracy of the estimation will be improved. The following relationship exists between the standard deviation of a population, σ, based on a large number of individual measurements, and the standard deviation, S, based on the means of groups of observations (Hoel, 1947, p. 65):

$$\frac{\sigma}{\sqrt{N}} = S \tag{4}$$

where N is the number of individual measurements in each group.

If the coefficient of variation is constant over a range of mean velocities, the average coefficient of variation can be substituted for the standard deviation in equation (4). This condition is approximately fulfilled for mean velocity values exceeding $\frac{1}{2}$ foot per second.

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[*]The coefficient of variation loses its significance as the mean velocity, V_m, approaches zero. For this reason velocities below 0.1 foot per second were arbitrarily omitted from this computation.

Suppose we wish to estimate the number of individual observations, N, that the mean must be based on so that the coefficient of variation is 25%. Substituting 91% for σ and 25% for S in equation (4) gives a value of N equal to approximately 13 measurements.

PREDICTION OF LONGSHORE CURRENTS.

The momentum approach to the prediction of longshore currents developed by Putnam, Munk, and Traylor (1949) was selected for use in this study. It relates the mean velocity V, of the longshore currents, to the wave height H, period T, angle of approach α, and slope of the beach i, according to the relation:

$$V = \frac{a}{2} \left[\sqrt{1 - \frac{4C \sin\alpha}{a}} - 1 \right] \qquad (5)$$

where

$$a = (2.61 \text{ Hi} \cos \alpha)/kT$$

and $C = \sqrt{2.28 \text{ gH}}$ is the wave velocity, k is the beach friction coefficient, and g is the acceleration of gravity.

The range of longshore current velocities obtained on the model and prototype beaches studied by Putnam, et al (1949), were individually limited and indicated that the beach friction coefficient "k" was relatively constant for a particular beach. However, the more recent observations in the San Diego area considered together with those of Putnam, et al (1949), indicate that "k" is a function of current velocity and cannot be considered a constant for a given beach. Using the tabulated observed values of longshore current, published in Putnam, et al (1949), the value of k was computed from equation (5) for all of their field and laboratory observations, and also for the series obtained in 1950 at the two beaches in the San Diego area by using the mean of the bottom longshore component.* The coefficient k is plotted as a function of the observed velocity in Fig. 8. Inspection of this figure strengthens the contention that the coefficient is a function of velocity; however, it should be mentioned that since k is not determined by direct measurement, it therefore not only reflects beach friction, but also any errors in measurement and any inadequacies of the theory.

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*The 1949 Torrey Pines Beach data (Table I A) was not used in this computation because of the inaccuracy in the measurement of the breaker angle. The transit-sighting bar method was not adopted for this purpose until the 1950 season.

The relation between the beach friction coefficient and the long-shore current velocity was obtained separately for the field data, the laboratory data, and the combined data by the method of least squares. Assuming a relationship of the form $k = bV^m$, the following were obtained:

For field data

$$k = 0.020 \ V^{-1.51} \tag{6}$$

For laboratory data

$$k = 0.029 \ V^{-1.54} \tag{7}$$

For all data

$$k = 0.024 \ V^{-1.51} \tag{8}$$

The close agreement between the above equations suggests that $k = 0.024 \ V^{-3/2}$ can be used as a good approximation for both laboratory and field observations. Since the type of bottom material represented in the equations (see Fig. 8) ranged from 1/4 inch pea gravel through sand to smooth concrete, the type of bottom apparently is not as important as the velocity in determining the value of k.

Substituting the value of $k = 0.024 \ V^{-3/2}$ into equation (5) gives the following relationship for the computed mean longshore velocity:

$$V_c = \left[\left(\frac{1}{4x^2} + y \right)^{\frac{1}{2}} - \frac{1}{2x} \right]^2 \tag{9}$$

where

$$x = (108.3 \ Hi \ \cos \alpha)/T$$

and

$$y = C \sin \alpha$$

As an aid in computing longshore currents, equation (9) is reproduced in the form of an alignment chart in Fig. 9. This chart gives the mean longshore current in feet per second, when the breaker height in feet, the period in seconds, the beach slope in per cent and the angle of breaker approach in degrees are known.

Using equation (9), the longshore current velocity V_c was computed for all of the field data listed in Putnam, et al (1949), and for the
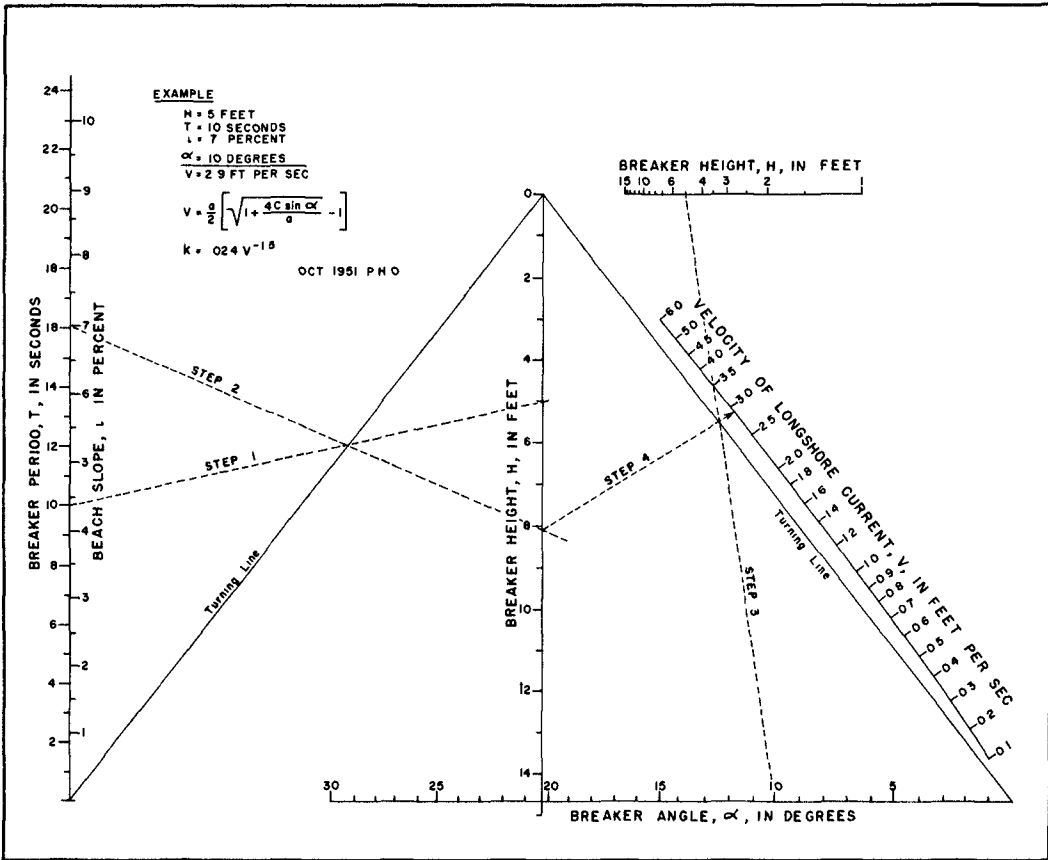
Fig. 9. Alignment chart for the computation of longshore current.
Procedure: (1) lay straight edge from appropriate T to H and deter-
mine intersection with turning line; (2) turn straight edge about
intersection to i, and determine intersection with H scale; (3) deter-
mine intersection on second turning line between H and α scale;
(4) align intersections of H scale and second turning line and read
velocity.

1950 observations at Torrey Pines and Pacific Beaches. The relative
error in percent, $E = 100 \dfrac{Vc-Vo}{Vo}$, between the calculated and the
mean observed velocity was then computed for each series of observa-
tions, and the values of E classified. From the classified values of
E the standard deviation of the errors (or standard error) σ_E, was
obtained by a relationship similar to equation (2).

σ_E was computed for (a) field data having an observed mean long-
shore current velocity above 0.1 feet per second, and (b) field data
having a mean velocity above ½ foot per second. The standard error
for these two cases was 54% and 41%, respectively.

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In the previous section on the variability of longshore currents it was found that the average coefficient of variation, C_v, for case (b) was 91%. The average coefficient of variation is not directly comparable with the standard error. However, the question can be asked, "How many individual measurements of current, N, must the mean velocity be based on in order that the average coefficient of variation be equal to the standard error obtained by computing the velocity from equation (9)?" By letting σ equal 91% and S equal 41% in equation (4) we find that N equals approximately 5 observations. Thus the errors in predicting mean currents above $\frac{1}{2}$ foot per second from equation (9) are comparable to the error of estimating the mean velocity from the mean of approximately 5 measurements.

Comparison of the values of the coefficient of variation and the standard error suggest that the number of measurements, N, must be greater than 5 for mean longshore velocities below $\frac{1}{2}$ foot per second, if the two errors are to be comparable. Also, to avoid station bias (see Fig. 7), the measurements should be obtained from many different stations scattered along the beach.

ACKNOWLEDGMENTS

REFERENCES

Forrest, D. R. (1950). A method of estimating wave direction, Bulletin of the Beach Erosion Board, vol. 4, No. 2, pp. 31-37.

Hoel, P. G. (1947). Introduction to mathematical statistics, John Wiley & Sons. 258 p.

Putnam, J. A., W. H. Munk, and M. A. Traylor (1949). The prediction of longshore currents, Trans. Amer. Geophys. Union, vol. 30, pp. 337-345.

Shepard, F. P. and D. L. Inman (1950). Nearshore water circulation related to bottom topography and wave refraction, Trans. Amer. Geophys. Union, vol. 31, No. 2, pp. 196-212.

Shepard, F. P. and D. L. Inman (1951). Nearshore circulation, Proc. First Conf. on Coastal Engineering, Council on Wave Research, Engineering Foundation, pp. 50-59.

Chapter 4

HYDRODYNAMICAL EVALUATION OF STORMS ON LAKE ERIE

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ABSTRACT

The wind velocities observed over cities on the southern coast of Lake Erie during storms are modified, according to recently given meteorological theories, to obtain the wind velocities coexisting over the surface of the lake water. On the basis of these reduced velocities and the associated wind tides of the lake, the coefficient of wind stress of wind action on the water of the lake is determined. Due attention is given to the fact that the form of the lake affects the relation between the wind stress and the associated wind tide. The coefficient of stress arrived at for the larger wind velocities is in substantial agreement with the values which Neumann determined for the Gulf of Bothnia. The matter of sea roughness is discussed briefly.

TREATMENT OF THE OBSERVED WIND VELOCITIES.

The determination of the coefficient of wind stress from a knowledge of total wind tides may be made with reference to the wind velocity at a standard height above the water surface of the lake. During severe storms, however, the winds are rarely measured over the lake, and the desired values of the wind velocities must be deduced from those observed over the cities on the borders of the lake. The data relating to storm effects on Lake Erie treated here are taken from the hydrographic charts prepared by the United States Lake Survey. These charts give the changes of water level at the lake extremities and also the movement and direction of wind for a few cities on the southern coast of the lake; namely, Toledo, Cleveland, Erie, and Buffalo. Where the wind data were lacking, the desired information was obtained from the Weather Bureau.

Three steps are involved in reducing the city wind velocities to the lake wind velocities. These will be discussed briefly. Now in the four cities mentioned above, the anemometer heights are not the same. Again, during the last half century in a given city, the anemometer heights have been changed from time to time. Therefore, the first step in the reductions is to change the values of all the observed wind velocities to the comparable velocities which would exist at a height of 165 feet above the ground level of the city. Let U_a be the wind velocity at the anemometer height z_a and U_2 the wind velocity at the standard height z_2. The Prandtl rule of velocities [1] ,

$$\frac{U}{U_*} = 5.75 \log \frac{z}{\epsilon_2} ,$$ (1)

where U_* is the shear velocity $\sqrt{\tau/\rho}$, and ϵ_2 the effective roughness of the cities, yields the desired relation

$$\frac{U_2}{U_a} = \log \frac{z_2}{\epsilon_2} \Big/ \log \frac{z_a}{\epsilon_2} . \tag{2}$$

In examining the distribution of the wind velocity over the cities, the question of the reference level from which heights are measured is important. It seems reasonable, in the absence of any special investigation of the matter, to suppose that if K is the average height of the city buildings, the reference level lies at a distance $K/2$ above the ground. As regards the effective roughness of the cities, one may follow Prandtl and put $\epsilon_2 = K/30$. The average height of buildings in an American city may be estimated to be 30 feet. Accordingly, the height z may be measured from a level 15 feet above the ground level of the city and the city roughness ϵ_2 may be taken as one foot. Owing to the logarithmic law of the distribution of velocities, the uncertainties in the values of the effective roughnesses are not serious.

Examination of the charts shows that the intensity of wind is very seldom constant along the southern coast of the lake at a given time of a storm period. Hence for the determination of the average effective wind a correlative relation must be assumed. In the writing of the relation two principles may be considered. First, owing to the shape of the lake, wind tides are related not to the absolute value of wind force but to the component of it resolved along the lake axis. Second, the forces being nearly proportional to the squares of the wind velocities, the averaging may be done by taking squares. In this process, however, the wind velocities of the cities on the border of the lake must be weighted. The method adopted is as follows. The total lake expanse is divided into three parts of equal extent. It is assumed that the winds at Toledo and Buffalo apply individually to the corresponding extreme parts. The middle part is divided further into two portions, the winds of Cleveland and Erie applying to these. On this basis the averaged effected wind is

$$U_2^2 = \frac{1}{3} \left[U_B^2 \cos(\theta_B - \alpha) + U_T^2 \cos(\theta_T - \alpha) \right.$$
$$\left. + \frac{1}{2} U_C^2 \cos(\theta_C - \alpha) + \frac{1}{2} U_E^2 \cos(\theta_E - \alpha) \right]. \tag{3}$$

Here, θ_B , θ_T , θ_C , and θ_E are the directions of the winds observed at the cities Buffalo, Toledo, Cleveland, and Erie. The quantities U_B , U_T , U_C , U_E are the wind velocities which should prevail over these cities 165 feet above the ground. The direction of the lake axis is given by α . The evaluation on the basis of eq 3 constitutes the second step in the reduction of wind velocities.

If U_1 be the velocity of wind over the lake at a height of 25 feet above the water surface, the third and the final step of the reduction is the use of relations to derive U_1 from the values of U_2 determined by the formula, eq 3. The relation between U_1 and U_2 is influenced by the sea roughness ϵ_1 and the city roughness ϵ_2 , and also by the magnitude of U_2 , as will be seen presently.

At this point the analysis of Rossby regarding the movement of air in the layer of frictional influence is very essential [2]. In this theory the layer is divided into two parts; the lower boundary layer and the upper layer. In the boundary layer the directions of wind velocity and stress vector are coincident. The angle between the wind in the layer and the gradient wind, ϕ_s, does not change with elevation since the deflective force arising from the earth's rotation may be neglected. The distribution of the wind velocities is given by the rule of Prandtl, eq 1. The height of this layer will be denoted by H and the velocity of the wind at this level by U_H. In the upper layer the directions of wind velocity and stress vector are no longer coincident. The angle between the direction of the wind in the layer and of the gradient wind changes with the elevation and tends to zero when heights are increased. According to Rossby the total height of the layer of frictional influence is about 13 times the height of the boundary layer.

Let H_1 be the height of the boundary layer over the water of the lake and H_2 that over the city on the lake borders. Let U_{H1} and U_{H2} be the limiting values of the wind velocities in these layers. As U_1 is the velocity of the wind prevailing at a height z_1 over the lake and U_2 is the velocity of the wind prevailing at a height z_2 over the city, the application of the law of velocities from eq 1 gives

$$\frac{U_1}{U_2} = \frac{U_{H1}}{U_{H2}} \left[\frac{\log \frac{z_1}{\epsilon_1}}{\log \frac{H_1}{\epsilon_1}} \div \frac{\log \frac{z_2}{\epsilon_2}}{\log \frac{H_2}{\epsilon_2}} \right] . \tag{4}$$

This may be written as

$$U_1 = M U_2 , \tag{5}$$

where

$$M = \frac{U_{H1}}{U_{H2}} \left[\frac{\log \frac{z_1}{\epsilon_1}}{\log \frac{H_1}{\epsilon_1}} \div \frac{\log \frac{z_2}{\epsilon_2}}{\log \frac{H_2}{\epsilon_2}} \right] . \tag{6}$$

For the evaluation of M as a function of ϵ_1, and U_2 the results,

$$H = 3.51 \times 10^{-2} \frac{U_g}{f} \sin \phi_s , \qquad f = 2 \Omega \sin \lambda, \tag{7}$$

$$U_H = U_g \left(\cos \phi_s - \frac{1}{\sqrt{2}} \sin \phi_s \right), \tag{8}$$

and

$$\log N = 1.694 \cot \phi_s - \log \sin \phi_s + 1.441, \quad N = U_g / f\epsilon, \qquad (9)$$

from the analysis of Rossby are sufficient. Here Ω is the angular speed of the earth's rotation, and λ the latitude. Using these relations, M can be evaluated as a function of the gradient wind U_g once the roughnesses, ϵ_1 and ϵ_2 are specified. In this it must be supposed that the gradient wind has the same intensity and the same direction over the lake and over a city on the border of the lake. Through the same relations U_2 may be expressed as a function of U_g. Hence M may be expressed as a function of U_2. Taking z_1 and z_2 as 25 and 150 feet, respectively, and ϵ_2 as 1 foot, a set of M values was computed for a set of assumed sea roughnesses. These are given in table 1.

Table 1
The effect of sea roughness on the relation between the wind velocities over lake and over city.

ϵ_1, cm	1.0	0.6	0.3	0.1	0.01
U_2 mi/hr	$M = U_1 / U_2$				
5	0.965	1.015	1.060	1.120	1.215
10	0.982	1.026	1.075	1.139	1.242
20	1.005	1.045	1.095	1.166	1.278
30	1.018	1.058	1.110	1.180	1.301
40	1.028	1.067	1.120	1.190	1.314
50	1.035	1.074	1.126	1.195	1.321
60	1.040	1.078	1.132	1.199	1.325

It is seen from the table that for a given wind velocity U_2 the value of M changes appreciably with the roughness of the sea. In the absence of previous information as to what should be the sea roughness of Lake Erie for the ranges of the wind intensities attained during severe storms, this roughness may be assumed provisionally and later tested. The value we have selected is $\epsilon_1 = 0.3$ cm. The corresponding M values may be read from table 1, and the formula to compute the effective wind velocity over the lake, from eq 3 and eq 9, is

$$U_1^2 = \frac{M^2}{3} \left[U_B^2 \cos(\theta_B - \alpha) + U_T^2 \cos(\theta_T - \alpha) \right.$$
$$\left. + \frac{1}{2} U_C^2 \cos(\theta_C - \alpha) + \frac{1}{2} U_E^2 \cos(\theta_E - \alpha) \right]. \qquad (10)$$

HYDRODYNAMICAL EVALUATION OF STORMS ON LAKE ERIE

The storms passing over Lake Erie may be divided into Westerlies and Easterlies. It is with the Westerlies that the greater wind tides are observed. Some nineteen important storms of this type are considered for this investigation. The hourly wind data for all these storms were treated according to the relations given by equations 2 and 10.

SUMMARY OF DATA ON WINDS AND WIND TIDES OF LAKE ERIE STORMS

The curves of the wind intensity $U^2 (= U_1^2)$ and of the total wind tides ΔH of the storm of December 31, 1911, shown in figure 1, are typical of all the storms used in the present study. Taking the presentation in figure 1 as an example, it may be seen that the entire manifestation of the storm may be broken into three epochs: the epoch of maturing storm, the epoch of relative steadiness, and the epoch of recession. The hydrodynamical behavior of the waters in the lake during these epochs is expected to show marked differences. In the initial epoch the wind must blow for some length of time before the response of the water to the action of the wind is completed. The reason for this condition is that any manifestation of wind tide must be associated with the flow of water from one end of the lake to the other. The action of the wind must establish a layer of drift current, the thickness of which must increase with time, either under the action of molecular viscosity or turbulent viscosity or both. As the water is being collected at the leeward end of the lake, a returning gravity current is created which likewise increases in intensity with time. The second epoch represents that steady condition in which the transport of water through the body of the drift current is counterbalanced by the transport of water in the returning gravity current that is maintained by the unchanging surface gradient of the lake waters. The third epoch is associated with decreasing wind intensity. With the strength of the wind reduced, the wind stress can not maintain the adverse gradient of the surface waters, with the result that a surge of water is produced. This is a wave motion with a period equal to the seich period of the lake. In some charts of the Lake Survey these seiches are clearly seen during the time of the falling storm. In the extreme epochs the inertia effects are very pronounced; in the intermediate epoch such effects are reduced considerably in value.

The water-level changes of the intermediate epoch have a direct bearing on the relation between wind velocity and wind stress. Referring once more to figure 1, the reference time is the instant of maximum wind tide. In the case of every storm considered, the reference time is obtained by taking the average of the times when the deflections at the individual ends of the lake are the largest. Quite often the maximum elevation of water at Buffalo occurs earlier than the maximum subsidence at Toledo. Total wind tide is the magnitude of the relative displacement of the water at the lake ends. The average value of the wind tide over a duration of four hours and around the reference time is taken as the total wind tide of the intermediate epoch. The average value of U^2 over a duration of five hours and immediately preceding the reference time is taken as the corresponding wind value. The observed total wind tides have been corrected for the barometric pressures.

The wind velocities and wind tides thus determined from every storm record are entered in table 2 with the dates of the storms indicated.

Table 2
Wind tides of Lake Erie during westerly gales.

No.	Date	ΔH feet	U Miles per hr.
1	Nov. 21, 1900	13.12	50.5
2	Oct. 20, 1905	6.68	31.4
3	Oct. 20, 1906	9.75	38.3
4	Jan. 20, 1907	12.04	48.1
5	Dec. 7, 1909	10.51	40.1
6	Dec. 31, 1911	9.53	38.9
7	Jan. 31, 1914	7.95	34.7
8	Dec. 9, 1917	10.17	43.2
9*	Dec. 9, 1917	4.56	33.1
10	Dec. 10, 1907	7.62	34.3
11	Dec. 18, 1921	12.30	45.1
12	Dec. 8, 1927	13.24	47.4
13	Dec. 9, 1927	4.13	27.2
14*	Dec. 9, 1927	3.44	26.1
15*	Dec. 9, 1927	1.73	22.1
16	Apr. 11, 1927	13.31	51.3
17	Jan. 22, 1939	9.40	38.8
18	Sept. 25, 1941	9.06	35.7
19	Jan. 2, 1942	12.53	40.4
20	Jan. 3, 1942	2.38	19.5
21	Nov. 22, 1946	8.36	34.2
22	Mar. 25, 1947	8.34	35.7

In some of the charts the storms show two peaks with an extended flat region between the peaks. The total tides of such regions are also included in the data of table 2 and are identified with asterisks. The graphic representation of the same data is given in figure 2.
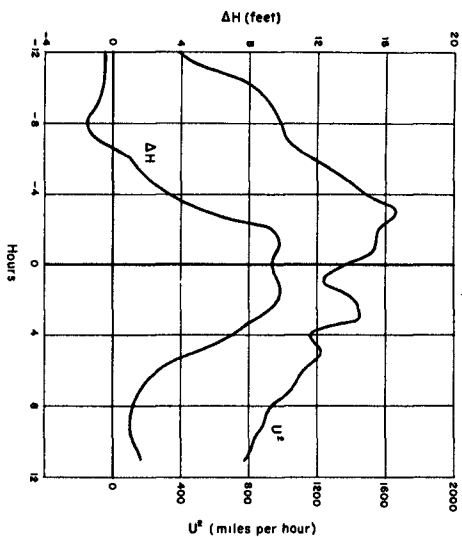
Fig. 1

Typical data of wind velocities
and the associated total wind
tides of a storm.
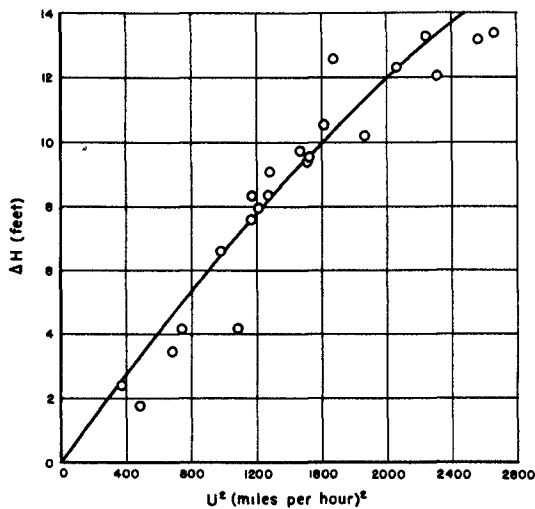
Fig. 2

The relation between total wind
tide and the wind velocities
for the Lake Erie storms.
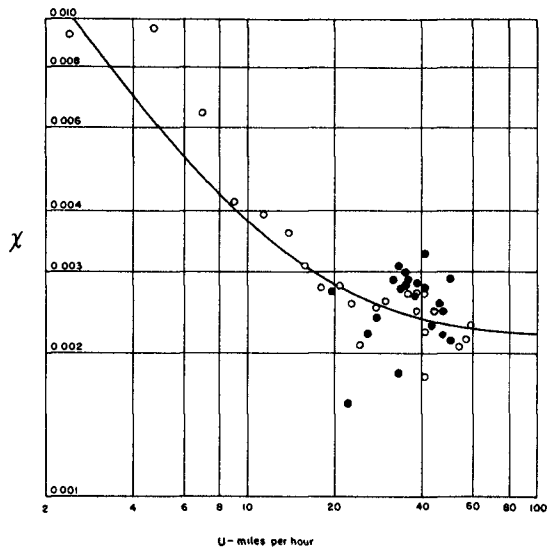
Fig. 3

The dependence of the coeffi-
cient of wind stress upon wind
velocities.
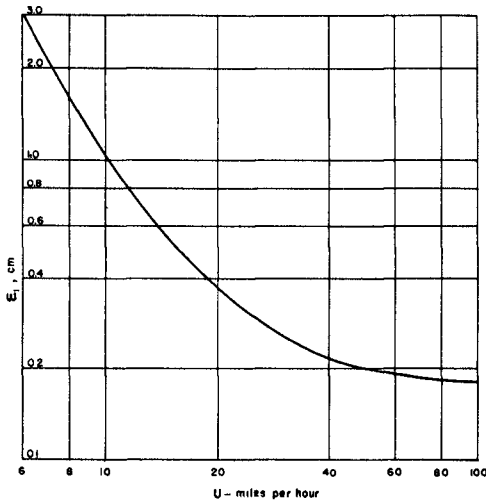
Fig. 4

The dependence of the roughness
of seas upon the wind veloci-
ties.

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WIND STRESS COEFFICIENTS OF THE LAKE ERIE STORMS

The wind-stress coefficient χ is defined·by the relation,

$$\tau_s = \chi \rho_a U^2, \tag{11}$$

where τ_s is the wind stress, ρ_a the density of air, and U the velocity of wind at some standard height above the water surface. In the present study the standard height selected is 25 feet. A change in the standard height implies a change in the value of the coefficient of stress. If χ is the stress coefficient based on the wind observed at the standard height z_1, and χ' that based on the wind observed at the standard height z_1', the coefficients are related by

$$\frac{1}{\sqrt{\chi}} - \frac{1}{\sqrt{\chi'}} = 5.75 \left[\log z_1 - \log z_1' \right], \tag{12}$$

a relation that is easily deduced by the application of the Prandtl law of velocities. If U is measured in centimeters per second, ρ_a in grams per centimeter cubed, τ_s is in dynes per square centimeter. If U is measured in feet per second, ρ_a in pounds per cubic feet, τ_s is in poundals per square foot.

The intermediate step in evaluating the stress τ_s from the total wind tide ΔH is a relation derived by solving the differential equation of wind tide subject to the condition of constant volume of water in the lake. Let $\bar{H}$ be the average depth of water in the section at x when the water of the lake is not disturbed. Let h denote the displacement of the water surface at x when the wind is acting. A simple dynamic consideration, after neglecting the small inertia effects of the flow, leads to

$$\frac{dh}{dx} = \frac{\tau_s + \tau_0}{\rho g (\bar{H} + h)}, \tag{13}$$

where τ_s and τ_0 are the stresses at the surface and at the bottom, respectively, ρ is the density of water, and g is the constant of gravitational acceleration. The origin of x may be placed at the windward end of the lake. As the volume of water in the lake is not changed, the condition of continuity is given by

$$\int_0^L B h \, dx = 0, \tag{14}$$

where B is the width of the lake surface and L is the length of the lake axis.

In natural lakes the mean depth of section $\bar{H}$ and the surface width B both vary with x. For these cases the mechanics of solution of eq 13 subject to the condition of eq 14 is greatly simplified when x is expressed in terms of L, and h and H in terms of H_o, the overall mean depth of the lake; B is expressed in terms of $\bar{B}$, the average width of the lake surface.

In solving the differential equation of wind tide in the case of a particular lake, $\bar{H}/H_o$ and $B/\bar{B}$ are expressed first as functions of x/L. Dispensing at this time with the details of the computations that were made for Lake Erie, we shall be content to give the final result. This result is

$$\frac{\tau_s + \tau_o}{\rho g H_o} \cdot \frac{L}{H_o} = 0.867 \frac{\Delta H}{H_o} - 0.134 \left(\frac{\Delta H}{H_o}\right)^2, \qquad (15)$$

which is a relation between the wind stress and wind tide as affected by the shape and the dimensions of the lake. The mean depth H_o may be taken as 58 feet and the length L as 272 miles. Writing

$$\tau_o = n \tau_s, \qquad (16)$$

and introducing τ_s from eq 11, we have

$$\chi = \frac{0.867}{1+n}\left[1 - 0.16 \frac{\Delta H}{H_o}\right] \frac{\Delta H}{H_o} \cdot \frac{\rho}{\rho_a} \cdot \frac{g H_o}{U^2}. \qquad (17)$$

At the present we are in the dark as regards the magnitude of stresses at the lake bottom. It will be supposed tentatively that the stress at the bottom is about one tenth as large as the surface stress. Thus putting $n = 0.1$,

$$\chi = 0.787\left[1 - 0.16 \frac{\Delta H}{H_o}\right] \frac{\Delta H}{H_o} \cdot \frac{\rho}{\rho_a} \cdot \frac{g H_o}{U^2}, \qquad (18)$$

which is the final form of the formula to evaluate the coefficient of stress when the total wind tide corresponding to a wind velocity is known. The formula applies to Lake Erie only and for winds moving from west to east.

Applying the above formula, eq 18, to the data of wind tides given in table 2, a set of values of the coefficient of stress are obtained, and these are shown in table 3. The same data are plotted as full circles in figure 3. There are plotted also in the same figure as open circles the values of the stress coefficients which Neumann obtains in an examination of the Pelman observations from the Gulf of Bothnia [3]. The original values given by Neumann were reduced to apply to a standard height of 25 feet above the water surface using the relation in eq 12.

Table 3
Coefficient of wind stress of Lake Erie gales

No.	$\chi \times 10^3$	U miles/hour	No.	$\chi \times 10^3$	U miles/hour
1	2.13	50.5	12	2.44	47.4
2	2.87	31.4	13	2.35	27.2
3	2.78	38.3	14	2.15	26.1
4	2.17	48.1	15	1.53	22.1
5	2.73	40.1	16	2.88	51.3
6	2.63	38.9	17	2.64	38.8
7	2.76	34.7	18	2.96	35.7
8	2.23	43.2	19	3.22	40.4
9	1.78	33.1	20	2.68	19.5
10	2.71	34.3	21	2.99	34.2
11	2.52	45.1	22	2.77	35.7

Examination of figure 3 reveals that in the region of higher wind velocities the coefficient of stress is practically independent of the wind velocity. Furthermore the coefficients of wind stress for the two areas are of like value. The average of the individual determinations equals 0.00254 in the case of Lake Erie and 0.00236 in the case of the Gulf of Bothnia. Now this comparison is for the high wind velocities. Significantly, the Gulf of Bothnia data for the small wind velocities show that the stress coefficient decreases with increasing wind velocities.

THE ROUGHNESS OF SEA

We shall adopt for the determination of the roughness of sea the procedure employed by Neumann. Writing eq 11 in the form,

$$\frac{U}{U_*} = \chi^{-\frac{1}{2}} ,$$

substituting in eq 1, changing z to z_1, and ϵ_2 to ϵ_1,

$$\frac{1}{\sqrt{\chi}} = 5.75 \log \frac{z_1}{\epsilon_1}, \qquad (19)$$

which relates the sea roughness ϵ_1 to the standard elevation z_1 and the coefficient of stress χ. In the present case since χ is determined from wind velocities which should prevail at a height of 25 feet above the water surface, then $z_1 = 672$ cm. As mentioned above, the average value of the coefficient of stress of Lake Erie for the wind velocities met in storms is 0.00254. This yields for the roughness of the sea the value $\epsilon_1 = 0.27$ cm. It will be remembered that in the computations needed to reduce the city wind velocities to the lake velocities, through the relation eq 10, the sea roughness had to be assumed provisionally to determine M appearing in the same equation. The assumed value was $\epsilon_1 = 0.3$ cm and this selection now appears to be satisfactory.

There is drawn a curve through the plotted points in figure 3. This curve may be utilized to determine the roughness of sea as a function of wind velocity for inland waters. The determinations by means of the formula, eq 19, putting $z_1 = 672$ cm are shown in the form of a curve in figure 4. The roughness of the sea decreases with increasing wind velocity. The decrease in the value of roughness is very slow for very strong winds. As the problem of sea roughness is yet an unsettled question, the statements above must be considered cautiously until confirmed by further observational data.

REFERENCES

1. Prandtl, L. (1932). Meteorologische Anwendung der Strömungslehre: Beitrage zur Physik der freien Atmosphare, vol. 19, p. 188.

2. Rossby, C.-G. and Montgomery, R. B. (1935). The layer of frictional influence in wind and ocean currents: Papers in Physical Oceanography and Meteorology, vol. III, no. 3, Cambridge, Mass.

3. Neumann, G. (1948). Über den Tangentialdruck des Windes und die Rauhigkeit der Meeresüberfläche Zeit. f. Meteor., Year 2, p. 193.

Chapter 5

WAVE-PRODUCED MOTION OF MOORED SHIPS

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SCOPE OF PAPER

This paper presents a brief description of some ship motion measurements made in Los Angeles Harbor in conjunction with the model study of inner mole at that location. It also includes the description of the battery of recording instruments which was developed as the result of the difficulties encountered in making the ship motion measurements by conventional means.

NEED FOR INFORMATION CONCERNING MOTION OF MOORED SHIPS

Knowledge of the characteristics of motion of moored ships is of considerable importance to several phases of harbor engineering. For example, it is a very important factor in general harbor design. It is becoming well recognized that wave motion and the resulting ship motion vary quite widely in different parts of a given harbor. Also the maximum acceptable motion of a moored ship depends upon the type of ship-to-shore operation that has to be carried out. For example, passenger and light cargo loading and unloading can be carried on successfully even though ship motions of relatively large amplitude are present. On the other hand, major operations at outfitting and repair docks and the operation of drydock gates require that ship motion be very small. Unfortunately, little quantitative information exists concerning either the maximum acceptable ship motions for the different types of harbor operation or the magnitude and other characteristics of the wave motion which will produce a given ship motion under a specified system of mooring.

The quantitative knowledge of ship motion is also of importance to the detailed design of all waterfront structures involving ship mooring since the maximum loading applied to such structures by the ship will commonly be as a result of the ship motion produced by wave action.

COMPONENTS OF SHIP MOTION

A moored ship generally has a very complicated pattern of motion. A convenient set of axes for analyzing this motion consists of a horizontal axis parallel to the pier or other mooring, a similar axis perpendicular to the pier, and a vertical axis, all passing through the center of gravity of the ship. When the various components of this motion are examined critically, it is found that they are not all of equal significance. Of the three linear components, the longitudinal and lateral are the most important since the vertical or "heaving" motion is nearly always of much smaller amplitude than the other two. Of the three angular motions, pitch seems to be the least important for the moored ship; whereas, roll and yaw may both cause considerable trouble. Pitching and heav-

ing are primarily induced by the vertical components of the wave motion. The other ship motions are the result of the horizontal water motion. It thus becomes obvious that the horizontal water motion is the fundamental cause of the significant components of motion of moored ships. The reason for this is that major ship motion is induced only by waves whose length is at least a large fraction of the ship length. The amplitude of the horizontal water motion of such waves is much greater than that of the vertical; hence, the horizontal motion of the moored ship is much greater than the vertical.

Unfortunately, the conventional method of measuring wave activity is by use of a water stage or wave height recorder. This measures only the vertical component and period of the wave motion. For long period waves this is a particularly insensitive method of measuring horizontal water motion. An added disadvantage is that the amplitude of the short period "wind chop" waves may be much greater than that of the long period waves that cause the ship motion. Thus, unless the wave height recorder is properly damped, its records may be meaningless. The wave height recorder suffers from still another disadvantage. A single instrument gives no information concerning the direction of travel of the waves. Due to the complicated nature of the wave patterns existing under natural conditions, it is often extremely difficult to determine the direction of wave travel from the simultaneous records of three wave height recorders spaced in a triangular pattern. However, if ship motion is to be correlated with water motion, it is necessary to know the direction of the horizontal motion of the water.

SUMMARY OF PROBLEM

To summarize the problem, it consists of two parts. The first part is to determine the linear and angular components of motion of moored ships and to evaluate the maximum amplitudes of the different components that can be tolerated for various types of harbor activities. The second part is to determine the magnitude and direction of the horizontal water motions and to correlate these measurements with the resulting ship motion for various types of ships under various mooring conditions. The harbor designer already has at his disposal at least two methods of predicting in considerable detail the horizontal water motion over an entire area of a proposed harbor. Many large harbors are designed with the help of model studies. If undistorted scale models are used, reliable measurements can be obtained of the horizontal water motion in various parts of the harbor. Furthermore, methods have been developed and are being improved constantly for the calculation of the wave pattern within harbors due to the wave energy entering through the breakwater openings. The missing links in the chain of design are the correlation of water motion with ship motion and the values of the allowable ship motion for the different operations.

SHIP MOTION OBSERVATIONS

Observations of ship movements at outfitting piers were made in the spring of 1944 as a part of the model study of the inner mole for the Naval Operating Base at Terminal Island in Los Angeles Harbor. The measurements were made as follows: large rectangular targets were fasten-
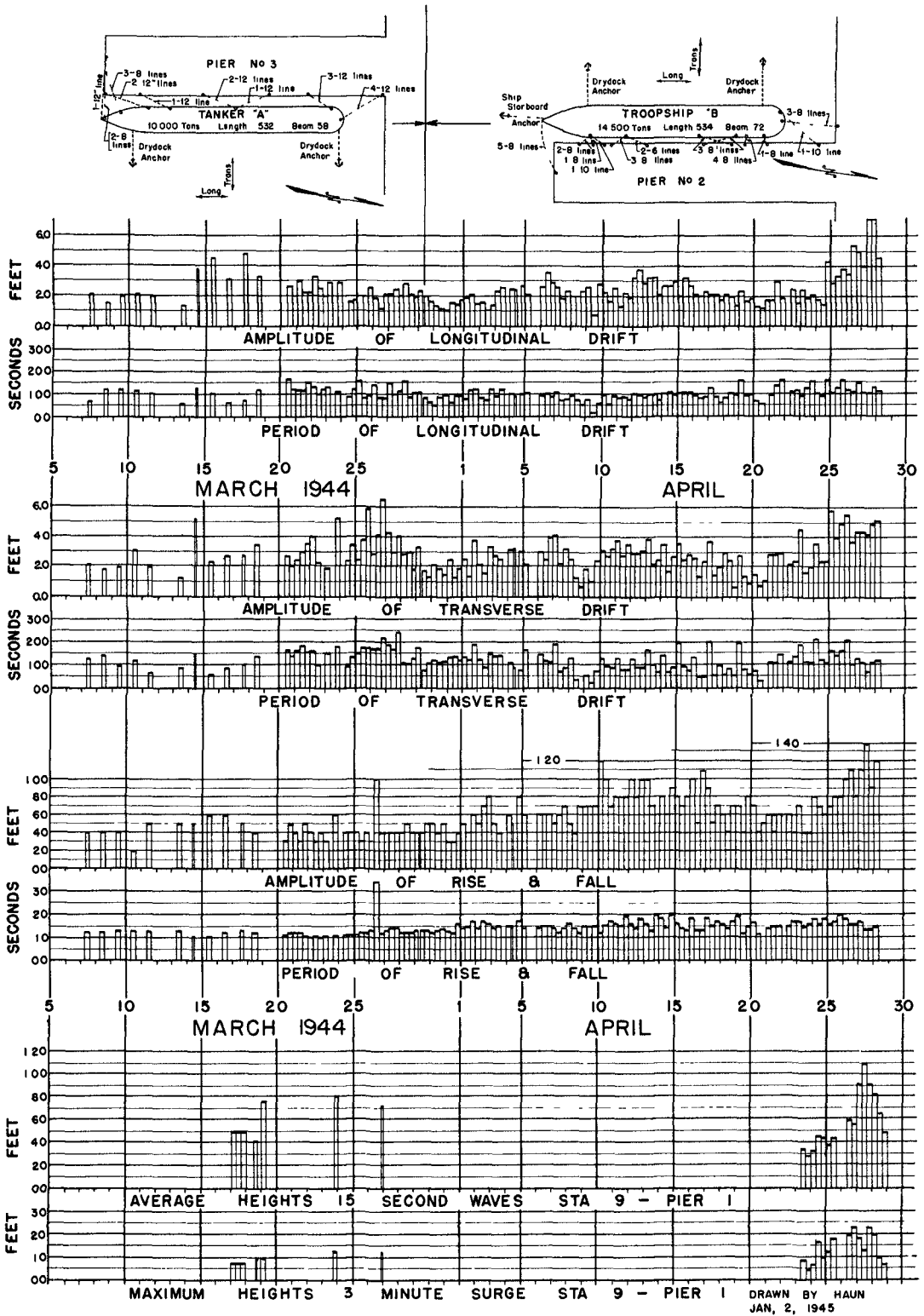
Fig. 1.   Observed ship and wave motions of tanker and troop ship.

ed to the bow and stern of the ship to be studied. These targets were
subdivided into numbered squares which were in turn subdivided into four
labelled quarters. Two surveyors levels were mounted in fixed positions
on the pier, one for each target. Determination of ship motion was made
hourly. The two targets were read simultaneously at very short intervals
until enough data had been obtained for the calculation of the amplitudes
and periods of the lateral, longitudinal, and vertical motions of the
ship. These three components were obtained from the original data through
the use of a plotting board and movable templates. This method proved to
have a satisfactory accuracy and be much faster than direct calculation.

These observations were continued over a period of about three months
and included nine different ships whose displacements varied between
10,000 and 14,500 tons. Most of the measurements were of little value
because the ship motion was too small to cause any difficulty in the
operation. However, on two ships, Tanker A and Troop Ship B, the measur-
ing period included some moderately high activity. Fig. 1 shows the ob-
served ship motion during these two sets of measurements together with
the wave height measurements as recorded by a float type wave height re-
corder operating in a stilling well.

GENERAL CHARACTERISTICS OF SHIP MOTION

At the top of Fig. 1 will be found a sketch showing the method of
mooring used for each ship. This figure requires detailed study. The
first thing to be noticed is that the wave height recorder shows two
types of waves, the normal 10-to-15 sec wind waves, and a long wave hav-
ing a period of approximately three minutes. The maximum observed wave
heights for the three minute waves is just slightly over .2 of 1 ft,
whereas the maximum height of the 15-sec waves was over 1 ft. During
much of the time the record indicates zero wave height. This does not
mean that there were no waves in the harbor, but simply that the wave
height records were not sufficiently accurate to detect a small ampli-
tude wave. The determination of the heights of the 15-sec waves was
complicated by the presence of wind chop. It proved to be very difficult
to get a satisfactory damping of the stilling well which would make it
possible to see the 15-sec waves and at the same time eliminate the
motion due to wind chop. Thus about .3 ft was the lowest 15-sec wave
that could be detected. It proved possible to measure heights of the
3-min waves somewhat more accurately, but here .05 ft was about the mini-
mum. These facts explain the ship motion shown on the record during
times in which no wave motion is indicated.

If the measurements of the vertical ship motion are next examined,
it will be seen that they correlate quite closely with both the period
and the amplitude of the 15-sec waves. (In this connection it should be
noted that throughout the diagram the word "amplitude" is used to mean
the double amplitude or total excursion of the ship.)

The transverse and longitudinal motions are in striking contrast to
the vertical. Both of these horizontal motions show large amplitudes and
long periods. Their periods are seen to average between 100 and 150 sec,
which is a reasonable correlation with the so-called 3-minute waves.

It may be concluded tentatively from these measurements that medium and large-sized ships move vertically with the same amplitude and period as the wave, provided that the wavelength is about equal to the ship length, or greater. It also may be concluded that the horizontal motion of such ships does not follow that of a short period wave, at least for wave heights of under 1 1/2 ft, but it does respond to the long-period waves.

HEIGHT OF WAVE PRODUCING DAMAGE

Fig. 2 is a record of the four days, April 23, 24, 26, 27, 1944, during which time there were periods of damage to ships and piers. The damage was all traceable to the horizontal motion of the ship. It will be observed that the periods of damage correlate well with those of the high amplitude 3-min. waves. Thus a similar record indicated that damage was present whenever the 3-min waves had a height of .2 of a ft or over. Rough calculations indicated that in the depth of the water existing at the piers, a 3-min wave with a .2 ft wave height would have a horizontal water oscillation of approximately 8 to 10 ft. This agrees very well with the observed longitudinal and transverse motions of the ship in spite of the fact that a relatively elaborate system of mooring had been used in an endeavor to minimize the motion.

Unfortunately, little can be said concerning the maximum tolerable movement at the outfitting piers. This is principally because at the time these measurements were taken, no means had been established for getting a reliable estimate of the effect of the motion on the work. There is one bit of evidence in that there was no complaint concerning the effect of the surge on April 23 and 24, whereas on April 26 the surge did interfere with the work. Thus it might be concluded that the horizontal motion of three to five feet, with a period of approximately 3 min, is about the upper limit of tolerance for general work.

INSTRUMENT DEVELOPMENT

The manual measurements described in the previous paragraphs were tedious, expensive, and not too satisfactory. Four men, two instrument men and two recorders, were required to take the simultaneous readings of the motion of the targets. For continuous readings, this meant a crew of 12 men for the three shifts. Despite this large crew, it was physically impossible to take enough readings to obtain the complete history of the ship movement. The work was also very discouraging to the crew because most of the time the ship motion was trivial. On the other hand, it was necessary to take the measurements continuously since the surge occurred without warning and often lasted only a few hours, which was too short a time to organize a crew and start taking measurements after the surge was first noted. Because of this situation, it was recommended in the report on the Terminal Island study that an effort be made to develop a battery of recording instruments which could operate with only infrequent service. The Bureau of Yards and Docks approved this recommendation and entered into a contract for the development of such a battery of instruments.

Fig. 3

Schematic installation of ship motion recorder.

Fig. 2

Observed motion of waves and of troopship "B" be-
fore and during period of damaging ship motion.

COMPONENTS OF INSTRUMENT BATTERY

As the result of the experience gained in the study just described, it was decided that the primary effort in the development of a ship motion meter should be to obtain measurements of the longitudinal and lateral motion of a ship with respect to the pier. The measurement of roll and yaw was considered to be desirable as a secondary objective. It was decided not to attempt to measure pitch. The close correlation between the vertical water motion of the waves and that of the ship found in the previous study indicated that a satisfactory solution would be to record the vertical water motion, but to omit attempting to measure the vertical motion of the ship. The set of measurements considered to be of equal importance to the determination of the horizontal component of the ship motion was that of obtaining the magnitude and direction of the horizontal water motion. Since the correlation of ship and water motions is one of the most important features of this type of study, it was decided that all of the measurements should be recorded simultaneously on a single record. The battery finally developed to meet these objectives consists of a ship motion meter which measures directly the longitudinal and lateral motions; an auxiliary attachment to this meter, which indicates roll and yaw; a bottom pressure recorder, which measures the vertical amplitude of the waves and surges and effectively eliminates the fluctuations due to wind chop; a current meter, which indicates magnitude and direction of the horizontal water motion; and a multiple element recording galvanometer.

Ship Motion Meter. Fig. 3 is a sketch showing the general scheme of the Ship Motion Meter. The meter itself is installed on the pier in a convenient location, either above or below the dock. It is connected to the ship by a very light cable (1/16-in dia.). An effort is made to locate the point of attachment to the side of the ship on a horizontal line running through the center of gravity of the ship and normal to the longitudinal axis. The line may be secured to the ship either by welding on a lug or by use of a small powerful permanent magnet. This latter method proved quite satisfactory since the force required to operate the instrument is very small. Fig. 4 shows three views of this instrument. The instrument consists basically of four potentiometers that are operated by the motion of the cable. The output of the linear potentiometer varies directly with the changing length of line connecting the instrument to the ship. The other three potentiometers are operated by the changing angle which the cable makes with the pier. The potentiometer cards are wound to give outputs proportional to the trigometric functions of the angle. Two of the potentiometers, one sine and one cosine, move with the horizontal angle that the line makes with the normal to the pier. Another cosine potentiometer measures the vertical angle that the line makes with this normal. The outputs of these four potentiometers are interconnected so as to give two outputs, one of which varies linearly with the longitudinal motion of the ship and the other varies linearly with the lateral motion.

The roll and yaw indicator is shown in Fig. 5. When this unit is used, it must be mounted on the ship in place of the simpler cable attachment. It consists of two more potentiometers, but in this case the output varies directly with the angle rather than with the trigonometric function. Fig. 6 shows a schematic diagram of the roll and yaw installa-

c. Cover plates removed.

b. Side view.

a. Front view.

Fig. 4. Ship motion recorder.
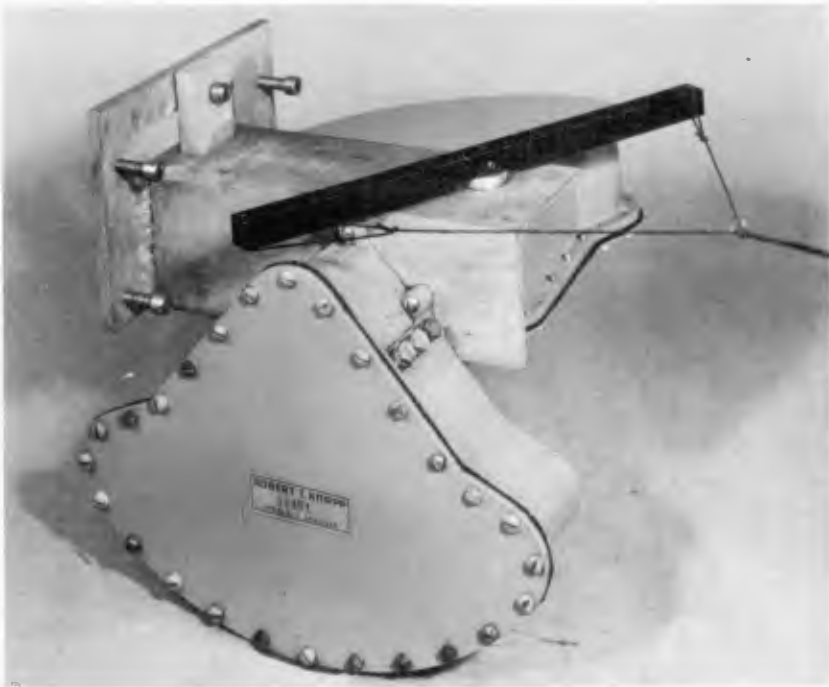
Fig. 5. Roll and yaw indicator.
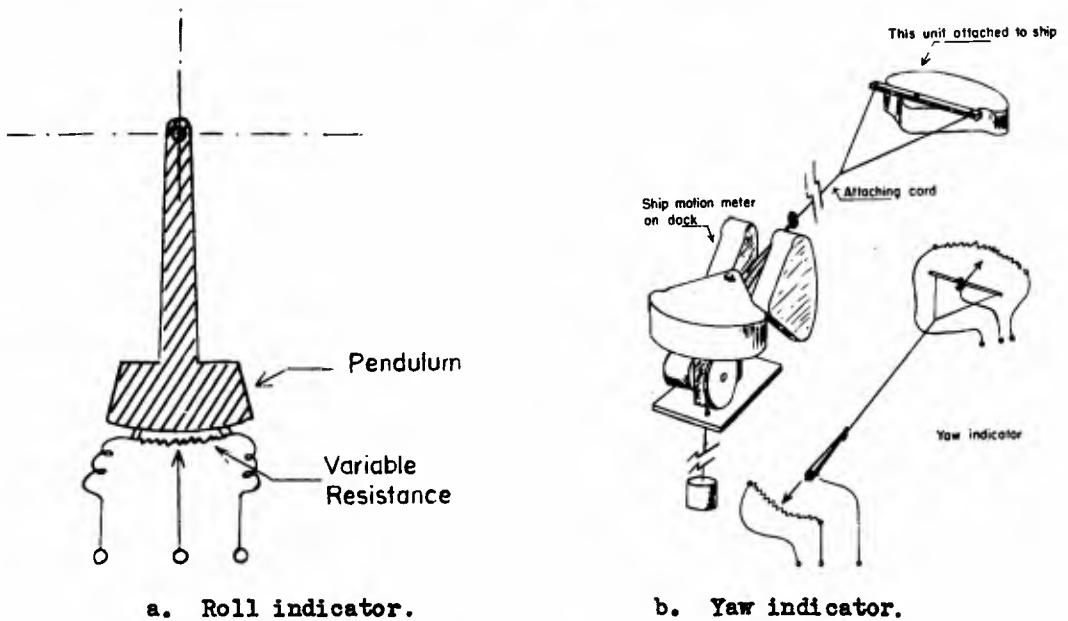
a. Roll indicator.   b. Yaw indicator.

Fig. 6. Schematic diagrams of roll and yaw indicator.
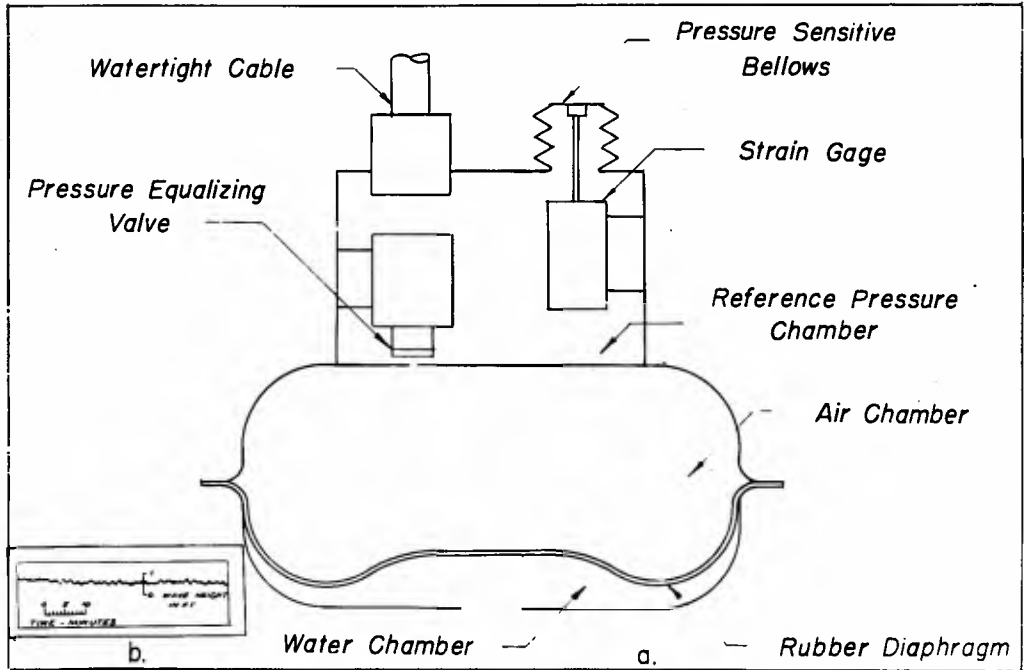
Fig. 7
Bottom pressure recorder, (a) schematic diagram, (b) record
made by recorder.

Fig. 8. Current meter mounted
in leveling tripod.
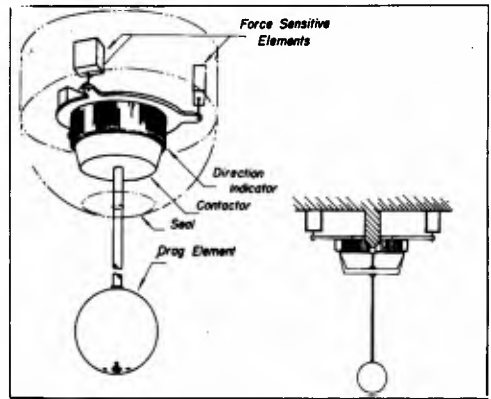
Fig. 9. Diagrammatic sketch;
construction of current meter.

tion. It is seen that the roll potentiometer is operated by a simple
pendulum. The yaw indicator requires an additional potentiometer to be
installed in the ship motion meter. This is also wound so that its out-
put is directly proportional to the angle. The two yaw potentiometers,
the one on the ship and the one on the pier, are connected electrically
so that the output is proportional to the difference between the two
angles. The output of the ship yaw potentiometer is proportional to the
angle between the instrument line and the ship, which is the sum of the
angle between the instrument line and the pier and the yaw angle of the
ship with respect to the pier. The output of the pier yaw potentiometer
is proportional to the angle of the instrument line with the pier. Thus
when this is subtracted from the ship potentiometer output, the remaining
output is proportional to the ship yaw angle.

Bottom Pressure Recorder. The Bottom Pressure Recorder is seen in
Fig. 7. The reason for designing a new instrument, rather than employing
one of the existing wave height recorders, was basically to obtain an in-
strument whose electrical output would be the same type as that from the
other instruments of the battery so that the same recorder could be used.
Furthermore, a relatively high sensitivity was desired so that low ampli-
tude, long period waves could be recorded, while at the same time the in-
strument was required to be undamaged by operation in water of widely
varying depth. The instrument is very simple, both in principle and in
operation. Fig. 7 shows a sectional diagram of the instrument. It will
be seen that it consists essentially of an air chamber, on one end of
which is a flexible metal bellows open on the inside to the air chamber
and surrounded by the water on the outside. This bellows is restrained
from moving in response to the pressure fluctuations in the water by a
rod which is connected to a wire-wound uncemented strain gage. The out-
put of this strain gage is thus directly proportional to the pressure
difference between the air chamber and the water. The air chamber is in
two parts, the upper, or working chamber, and the lower, or depth compen-
sating chamber. The upper chamber has a constant volume, but the bottom
of the lower chamber is a very flexible rubber diaphragm which is directly
in contact with the water. There is a single connection between the two
chambers through a pressure-equalizing valve. This is a solenoid valve
which is normally held tightly closed by spring pressure and is opened
only when the solenoid is energized. It will be seen that when the e-
qualizing valve is open, the pressure in the working chamber will be that
of the surrounding water. During installation of the instrument in a new
location, the equalizing valve is held open while the instrument is being
lowered to its operating location. The valve is then closed and the in-
strument immediately begins to function, indicating the difference in
pressure between the working air chamber and the surrounding water as the
latter fluctuates due to the passage of the waves.

Horizontal Current Meter. The current meter, together with its
leveling tripod, shown in Fig. 8, is designed to give a record of the
magnitude and direction of the horizontal water motion. Since the re-
cord is continuous, it also indicates the period of the oscillations.
The actuating element of the meter consists of a sphere suspended below
the case by means of a rod. The rod is pivoted within the case. The
freedom of motion required for operation is obtained by means of a flex-
ible rubber unit which also acts as a seal. The force of the water mov-
ing past the ball tends to cause the upper end of the rod to rotate in-
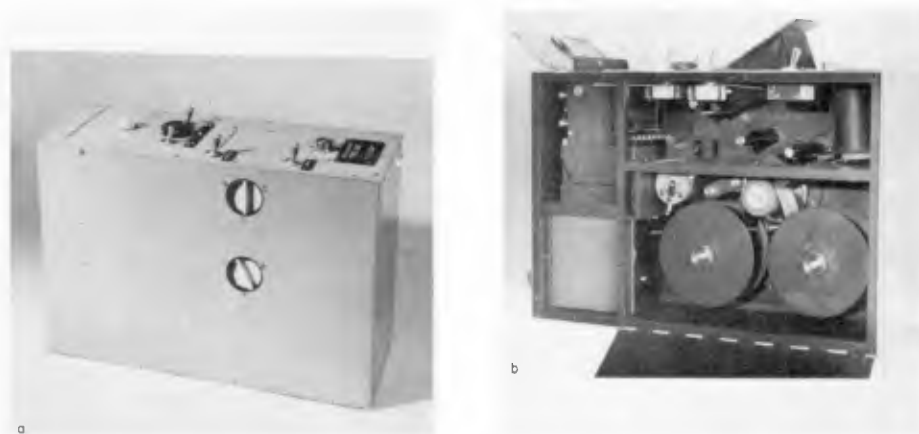
Fig. 10. 35 mm. recording galvanometer; (a) closed, (b) open.
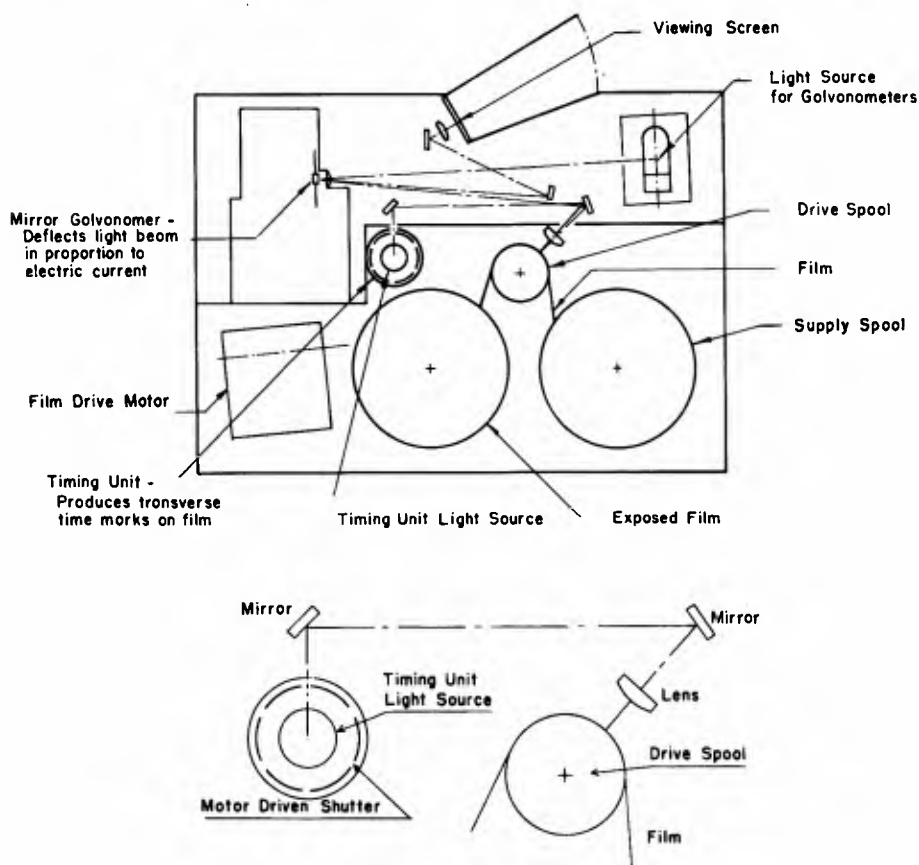
Fig. 11. Diagram of internal construction of recording galvanometer.

side the case. This rotation is prevented by a battery of three wire-
wound, unbonded strain gages similar to the one used in the Bottom Pres-
sure Recorder. These gages are connected electrically so that their com-
bined output is proportional to the horizontal force on the ball inde-
pendent of the direction from which this force is applied. Interposed
between the rods and the set of strain gages is a $360°$ wire-wound po-
tentiometer. A rigid disc carrying a special ring contact is fastened to
the rod. The diameter of the ring contact is that of the potentiometer
winding, and as the rod starts to move under the influence of the water
force the ring makes contact with the potentiometer at one point on the
circle. Since this point is determined by the direction from which the
water force comes, the potentiometer output is proportional to the direc-
tion of the water motion. The instrument operates completely filled with
oil and the case is provided with a small rubber bellows which equalizes
the pressure with that of the surrounding water. Thus the instrument is
capable of operating at any depth without modification. Fig. 9 is a dia-
grammatic sketch of the internal construction of the current meter. Since
it was proposed to install this meter on the natural harbor bottom, a
special mounting tripod was designed to support it. It is desirable to
have the ball hang vertically below the meter so that the readings of the
meter will be proportional to the horizontal water motion. Therefore, a
leveling device was incorporated in the tripod head. Furthermore, if the
meter is lowered in open water, it is necessary to know the orientation
of it with respect to the cardinal directions. Therefore, an aircraft
type transmitting compass was installed on the tripod in addition to the
leveling mechanism. The installation was completed by the addition of
an electrically operated level indicator.

The Recording Galvanometer. Although several excellent multiple-
element recording oscillographs are available commercially, they are all
designed primarily for use in investigating relatively high speed phe-
nomena. Although it is possible to reduce the film speed considerably,
it is difficult to accomplish the very large reduction that would be re-
quired to obtain a three-week continuous record without reloading, which
was the goal established for this application. Also it was felt desir-
able to use 35 mm film in place of the much wider recording material nor-
mally employed in the standard oscillographs. Therefore, a special re-
cording galvanometer was constructed utilizing a standard 9-element gal-
vanometer block from one of the commercial oscillographs. Fig. 10 shows
two views of the completed instrument and Fig. 11 a diagram of the in-
ternal arrangements. The timing unit gives a two-minute timing line on
the film. This film is standard, unperforated, dye-backed microfile film
which permits daylight loading and unloading and operates for three weeks
on a 100-ft spool.

OPERATION OF THE INSTRUMENT BATTERY

The instruments described were completed and operated individually
and as a battery for sufficient time to demonstrate their satisfactory
performance. Unfortunately, however, it had not yet been possible to in-
augurate a program of study with them, either to determine the limits of
tolerable motion for various harbor activities or to ascertain the corre-
lation between ship and water motion for various systems of mooring.
These are both extremely important steps and it is hoped that this work
can be continued along these lines.

ADDITIONAL INFORMATION NEEDED

There is still another step that needs to be taken to complete the information necessary for the design of waterfront structures to which ships are moored. This is to determine the forces produced on waterfront structures by the motion of moored ships. A program of direct measurement does not seem to be indicated for this step, or at least not until a great deal more information is available concerning the correlation of water and ship motion. The basic reason for this statement is that there are so many variables that affect the actual forces between the pier and the ship that it would require a very large and lengthy program of field measurements to cover enough different cases to make such empirical information useful and reliable. On the other hand, it may not be necessary to determine these forces by direct measurements after a thorough understanding has been obtained concerning the ship motion produced by given waves. When this information is available, it should be possible to calculate the resulting forces with reasonably good accuracy and by methods which would take into account the geometric and dynamic characteristics of the ship as well as the system of mooring and the wave characteristics at the location of the given structure.

REFERENCES

Knapp, Robert T. (1951). Determination of Wave, Surge, and Ship Motion, U. S. Naval Station, Long Beach, California. Final Report of Research Program under Contract with Bureau of Yards and Docks, U.S. Department of the Navy.

Knapp, Robert T. and Vanoni, Vito A. (1945). Wave and Surge Study for the Naval Operating Base, Terminal Island, Calif. Hydraulic Structures Laboratory, California Institute of Technology.

Wilson, Basil Wrigley. (1950). Ship Response to Range Action in Harbor Basins. Proc. ASCE. Vol. 76. Separate No. 41.

COASTAL ENGINEERING

Chapter 6

CHANGES IN SEA LEVEL DETERMINED FROM TIDE OBSERVATIONS

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Heights on land as well as depths in the sea are measured with reference to the surface of the sea or sea level. But the level of the sea is constantly changing under the influence of the rise and fall of the tide and of varying meteorological conditions. For that reason use is made of an average level called mean sea level, this datum being derived from tide observations.

The concept of mean sea level is very simple. The surface of the sea is at all times disturbed by one cause or another and mean sea level at any point is simply defined as the average or mean level of the sea at that point. This concept of mean sea level, by implication, makes its determination appear a simple matter. For all we need do to determine mean sea level at any place is, apparently, merely to measure the changing level of the sea at that place for some time and derive the average value. But when we are actually confronted by the necessity for a precise determination, the problem is found to be not quite so simple.

To begin with, it must be noted that in determining mean sea level at any point, we are determining not a scale reading on an instrument with a fixed zero, but the location of a horizontal plane on the earth. For measuring the changing level of the sea we make use of a tide staff, which is merely a rod or staff graduated in feet and decimals, fixed in a vertical position. How far below the surface of the sea the zero of this staff is placed is wholly arbitrary. This means that the tide staff reading has meaning only when related to some fixed point on land. Ordinarily the tide staff is fixed to the face of a pile or wharf, but to make sure the tide staff can always be maintained at a fixed elevation, it must be referenced to a number of adequate bench marks on shore. This is an extremely important matter which in the past has not been carefully observed.

The most effective method for measuring the changing level of the sea is through the use of an automatic tide gage which makes a continuous record of the rise and fall of the surface of the sea. From this record we can determine the height of the sea at each hour of the day and from these hourly heights we can derive the average height of sea level for a day, a month, a year, or for as long a period as may be desired.

The question that immediately arises is, for how long a period of time must we average the changing level of the sea to arrive at a determination of mean sea level? Since the tide is the predominating cause of the fluctuation of the surface of the sea, and since the primary period of the tide is a day, it is obvious that if we average the hourly heights of the tide over the period of a day we will eliminate the tide and derive a first approximation to sea level.
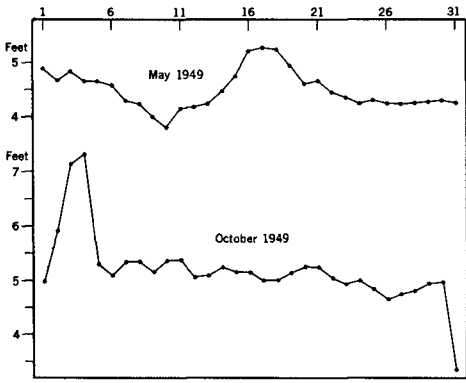
Fig. 1
Daily sea level, Galveston,
Texas, May and October, 1949.
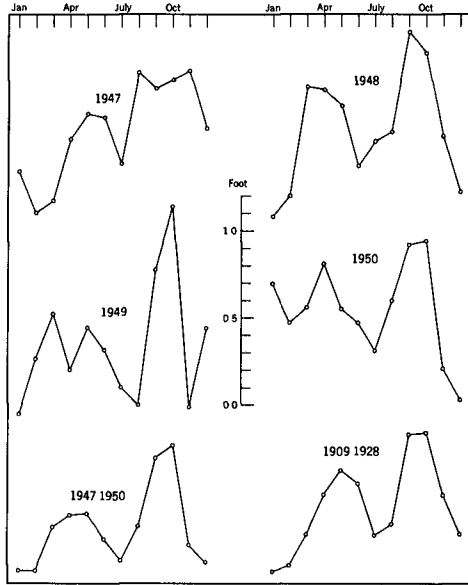
Fig. 2
Monthly sea level, Galveston,
Texas.
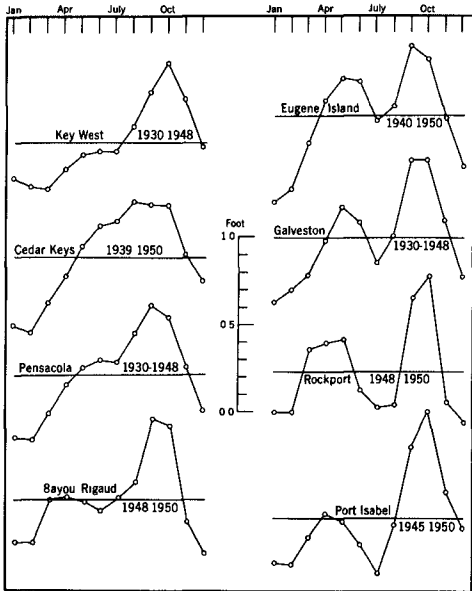
Fig. 3
Seasonal variation in sea
level, Gulf coast.
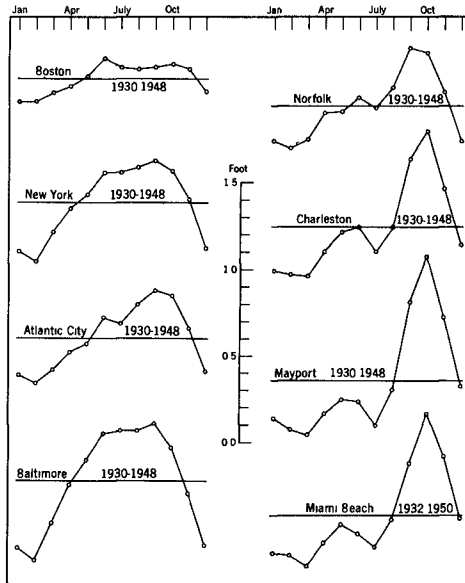
Fig. 4
Seasonal variation in sea
level, Atlantic coast.

On investigation it is found that sea level at any place changes from day to day. In Figure 1 there are shown the daily sea levels at Galveston for the two months of May and October 1949. The heights are referred to the tide staff at the tide station. For May the change in sea level from day to day was generally less than a tenth of a foot, though at times it was several tenths of a foot. During this month sea level on the 17th was $1\frac{1}{2}$ feet higher than on the 10th.

In October weather conditions were not as uniform as during May and greater variations in sea level from day to day occurred. For example, on the 5th, sea level was 2 feet lower than on the 4th. And during this month, sea level on the last day was nearly 4 feet lower than on the 4th.

Sea level thus varies considerably from day to day, and a determination of sea level from one day of observations, even in calm weather, can give only a very rough approximation to mean sea level. Clearly, therefore, to determine mean sea level with any pretense to precision, a longer series of observations is necessary, and a month suggests itself as a possibly desirable period.

In Figure 2 are shown the monthly heights of sea level at Galveston as measured on the tide staff for each of the four years 1947 to 1950. Each monthly height represents the average of some 700 hourly heights, and as might be expected, sea level from month to month differs less than from day to day. Generally the differences from one month to another are one or two-tenths of a foot, though occasionally they may be as much as a foot. Within a year the lowest and highest monthly values of sea level may differ by more than a foot.

A glance at these diagrams shows that sea level at Galveston for these four years was generally low in the winter months and high in the fall months, with a secondary maximum in spring and a secondary minimum in July. If we take the average of the sea level heights for corresponding months of the four years, we derive the curve shown in the lower left of Figure 2. This strongly suggests a seasonal variation in sea level. In the lower right of Figure 2 is shown the average monthly heights of sea level at Galveston for the 20 year period 1909-1928 and this is seen to approximate closely the curve derived from the four years 1947-1950.

Now a seasonal variation in sea level is a characteristic feature of the sea everywhere, but it has distinctive local, or perhaps more accurately, regional characteristics. In Figure 3 is shown the curves of seasonal variation of sea level at eight stations on our Gulf coast, based on various years of observations. The horizontal line on each diagram represents the height of mean sea level at each station for the period of observations indicated. Each place has its characteristic curve of variation, but places relatively near each other show much the same pattern. Thus, Key West, Cedar Keys and Pensacola resemble each other; likewise Eugene Island and Galveston, Rockport and Port Isabel.

It is of interest to note how the pattern from a single minimum and a single maximum, changes gradually from the eastern end of the Gulf to
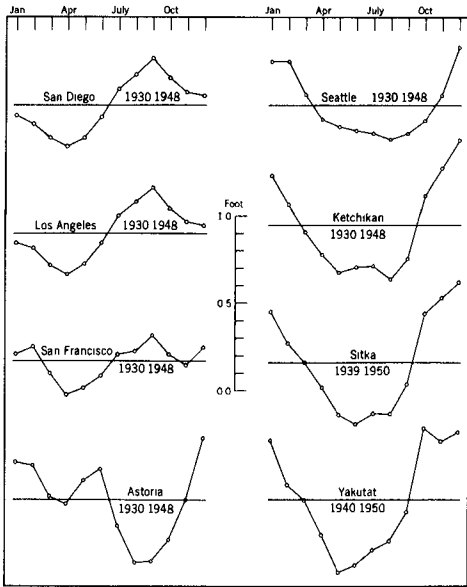
Fig. 5
Seasonal variation in sea
level, Pacific coast.
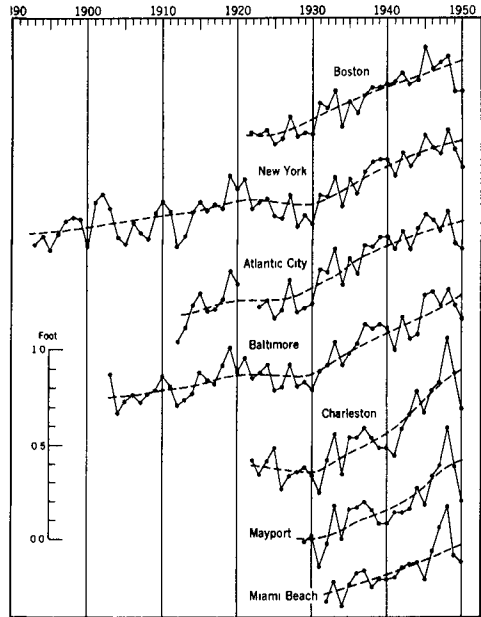
Fig. 6
Yearly sea level, Atlantic
coast.
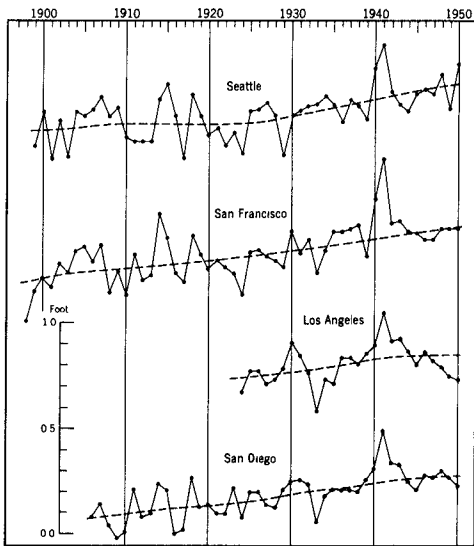
Fig. 7
Yearly sea level, Pacific
coast.
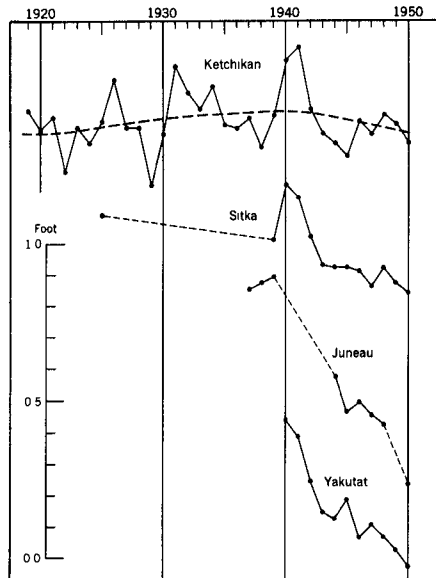
Fig. 8
Yearly sea level, Alaska

the double maxima and minima. At Key West there is barely an inkling of this; at Pensacola there is a beginning, but at Eugene Island, west of the Mississippi River it is well developed and continues to the western end.

If we examine the seasonal variation of sea level on the Atlantic coast of the United States, shown in Figure 4, we find again distinctive patterns which are characteristic for relatively large areas. Thus New York, Atlantic City and Baltimore show much the same pattern with a single maximum and minimum. Charleston, Mayport and Miami Beach, on the other hand have also a secondary maximum and a secondary mimimum.

In Figure 5 are shown the seasonal variations in sea level on our Pacific coast. Again, distinctive regional patterns, differing in the different regions. The seasonal variation in sea level is thus a characteristic feature of sea level throughout the world.

In view of the seasonal variation in sea level, it follows that a single month will not determine mean sea level accurately. A year immediately suggests itself as a desirable period, for within a year, the seasonal variation in sea level is eliminated and the effects of wind and weather likewise tend to balance out.

In Figure 6 are shown the yearly values of sea level at a number of our Atlantic coast stations. Each yearly height, represented by a small circle is the average of nearly 9,000 consecutive hourly heights.

Disregarding for the present the dashed-line curve associated with the diagram for each station, it is seen that sea level varies from year to year, generally by something like several hundredths of a foot but occasionally it may be as much as 0.2 foot. Furthermore, in the past 20 years there appears to have been a progressive rise in sea level. In the 20 year period from 1930 to 1950, the yearly sea level along the Atlantic coast for 1948 was about half a foot higher than the yearly sea level in 1930.

The fluctuations from year to year are in large part due to the disturbing effects of wind and weather; but if we smooth out these fluctuations - as is done by the dash-line curves - it is seen that there has been a steady progressive rise in sea level along the Atlantic coast which has averaged 0.02 foot per year.

The fluctuations in sea level from year to year are not confined to the Atlantic coast. In Figure 7 are shown the results from our Pacific coast tide stations. The fluctuations from year to year are of about the same character as on the Atlantic coast, but the progressive rise in sea level in the past two decades is only about 1/3 that on the Atlantic coast.

For the coast of Alaska our observations do not cover many years. But they are very interesting in their indications, for here sea level is falling. In Figure 8 the data for four stations are shown. At Ketchikan, which is the southernmost tidal station in Alaska, it appears that sea level

was slowly rising till about 1940, and since that
time sea level has been falling slowly. At Sitka,
Juneau and Yakutat, the series are too short for
definite quantitative evaluation, but the evi-
dence decidedly indicates a fall in sea level.

In this connection tide observations made
at Skagway, Alaska are of interest. From 1909
through 1911 we had a three year series of tide
observations at that place, reference to bench
marks. In 1944 the tide station was reestab-
lished and we find that in the intervening period
of about 35 years, sea level there fell about $2\frac{1}{2}$
feet.

Fig. 9. Yearly sea
level, Gulf coast

In Figure 9 are shown the yearly sea levels
at four stations on the Gulf coast. At Key West
the features are much the same as on the Atlantic
coast - little change until 1930 and a rise since
that time of about 0.02 foot per year. At Cedar Keys there was an inter-
ruption in the observations between 1925 and 1939, but the 10 years from
1915-1925 indicate little change in sea level, while from 1939 the rise
is at a rate somewhat greater than at Key West. At Pensacola the rise in
sea level is at a more rapid rate in recent years, the change appearing
to have occurred about 1940.

For Galveston the results are especially interesting since a contin-
uous series of 40 years is available. During this time sea level has
changed by almost exactly one foot, but the results clearly indicate a
change in rate of rise between 1937 and 1938. From 1909 to 1937 the rise
was at the rate of about 0.015 foot per year, while since that year the
rise is at the rate of about 0.05 foot per year or more than 3 times the
previous rate.

It is convenient to speak of the rise or fall of sea level because
the observations are made on a tide staff fixed to the land. But obvi-
ously, to say that sea level has risen with respect to a fixed point on
the shore, is only another way of saying that the fixed point has sub-
sided with respect to sea level. To determine which is the active agent
and which the passive is another problem. If the coast is the active
agent, the subsidence is absolute; if sea level is the active agent, the
subsidence is relative; but in either case there is a lowering of the
coast relative to sea level. What we have been trying to show is that
tide observations are furnishing data which permit quantitative determi-
nations of changes in the relative elevation of land to sea.

One other matter should perhaps be mentioned. The tide rises and
falls from sea level. If at any place sea level is changing it means
also that the datums of high water and low water are changing by the
same amount as sea level.

COASTAL ENGINEERING

Chapter 7

SALINITY PROBLEMS

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INTRODUCTION

The basic sources of salt-water pollution are the ocean, industry, and the soil. The ocean is responsible for the intrusion of salt water into rivers, canals, and lakes, and for infiltration of sea water into aquifiers which are tapped by wells. Industry causes salt-water pollution by discharging the brine of mines, oil wells, tanneries, and other industrial wastes into rivers and lakes. The soil is a source of salt-water pollution because of the run-off from chloride-bearing soils and the solution of soluble rocks. The most common and important source of salt-water pollution is the ocean, and is the only source considered in this paper.

NATURE OF SALINITY PROBLEMS

The types of salinity problems encountered are too numerous to permit coverage in a single paper; however, the most important are those pertaining to pollution of municipal, industrial, and irrigation water supplies, and to shoaling of navigation channels and harbors because of precipitation of suspended or dissolved solids which would otherwise pass out to sea. The subsequent parts of this paper define the above problems in more detail, describes methods which have been proposed or adopted for the solution of similar problems, and describes certain hydraulic model investigations of salinity problems. A selected bibliography is included which presents references to the most important literature available to the writer. References listed in the bibliography can be obtained on loan from the Research Center Library, Waterways Experiment Station, Vicksburg, Mississippi.

Pollution of water supplies

Salt water does not constitute a health hazard, but its taste in drinking water is very objectionable. The USPH "Treasury Standard" therefore limits the amount of chlorides to 250 parts per million, which is the approximate concentration at which consumers will begin to complain of salty taste. Many industries require water of very low salinity, the maximum allowable in certain products being 10 to 15 parts per million of chlorides.

Problems relative to pollution of irrigation water supplies are generally similar to those of municipal and industrial supplies; however, the demand for irrigation water and the tolerable limits of salinity vary with the season of the year and the crops to be irrigated. The results of a study of the Pecos River Basin by the National Resources Planning Board indicated that: (a) for salinities up to 3000 parts per

million total salts little injury to crops was evident; (b) between 3000 and 600u parts per million growth was limited to salt-tolerant crops; (c) between 6000 and 10,000 parts per million salt-tolerant crops will grow but seldom thrive well; and (d) above 10,000 parts per million plant growth was limited to some grasses and shrubs.[1]*

The results of a study made by the Louisiana State Department of Conservation to determine tolerable limits of salinity in irrigation water for rice indicate that the allowable limit ranged from 560 to 2900 parts per million total salts, depending upon the stage of growth of the crops.[2] In the Sacramento-San Joaquin Delta in California, the tolerable limit was established as approximately 330 parts per million chlorides. Crops irrigated in this area consist primarily of asparagus, potatoes, sugar-beets, corn, beans, and other truck crops, and considerable acreage in fruit orchards. It is therefore evident that a tolerable limit of salinity for all irrigation supplies can not be established. The crop to be irrigated, the characteristics of the soil, the amount of irrigation water necessary, and the stage of growth of the crop all affect the salinity that may be used without harmful effects. Only a tnorough study of local conditions, and experimental determination of allowable limits of salinity for all stages of growth of the crop to be irrigated, will indicate whether or not the water available is suitable for irrigation purposes.

Shoaling in rivers and harbors

A large part of the silt carried by some rivers is in the form of collodial or semi-collodial suspension or in solution. An important property of these particles of solids is that they exhibit no tendency to ball together and form deposits on the river beds, for the reason that each particle is charged with a negative electric potential. This potential being the same in all particles, the latter repel each other and a complete state of dispersion prevails so long as the water is fresh. Contact with salt water, however, causes a base-exchange reaction, whereby the electric potential is neutralized and a process of clotting, technically known as coagulation or flocculation, results. At first the flocs or lumps of coagulated material are quite small; however, as more and more particles are attached the lumps attain sufficient size and weight to sink to the bottom, thus effecting deposits on the river beds. Shoaling from this source appears to be most serious in the tidal sections of rivers draining from the Appalachian Mountains and discharging into the Atlantic Ocean and the eastern portion of the Gulf of Mexico.[3]

MANNER OF SALINITY INTRUSION

Intrusion of salt water from the ocean may occur as flow through open channels or by infiltration into ground water supplies. In many coastal areas the source of municipal and other water supplies consists of wells. In some cases such wells have been driven indiscriminately, and are being pumped at a rate greater than that at which the acquifer

*See references at end of chapter

is being replinished with fresh water. Under these conditions salt
water may be drawn into the water-bearing strata and cause contamination
of the entire supply. Occasionally, the ground water supplies of
coastal areas may become polluted because of surface flooding by sea
water. Many wells in New England were contaminated in this manner as a
result of the 1943 hurricane; however, the contamination was only
temporary, and continued pumping of the wells soon reduced their
salinities to normal levels.[4]

The upstream movement of salt water in open channels is attribu-
table to the greater density of salt water as compared to that of fresh
water. The nature and extent of such intrusion in a specific channel is
dependent upon the range of tide and salinity of the sea water at the
mouth of the estuary, the physical and hydraulic characteristics of the
estuary proper, and the volume of fresh water being discharged into the
estuary from the fresh-water section upstream. The manner of intrusion
may vary from the well-defined wedge which is typical of the lower
Mississippi River to the salinity front which may be found in San Francisco
Bay and Delaware Bay and River; likewise, any modification between these
two extremes is possible if the proper physical and hydraulic conditions
exist in an estuary.

The well-defined salt-water wedge, such as exists in the lower
Mississippi River, is illustrated by Fig. 1. The range of tide at the
mouth of the Mississippi River is so small that there is no reversal of
flow in the river because of tidal action; therefore, the position of
the wedge is governed almost entirely by the volume of fresh water flow.
Flow within the salt-water wedge is always upstream because of its
greater density, while flow in the fresh or semi-fresh water stratas
above is always downstream. The interface between the salt and fresh
water is fairly well defined, and along this interface the fresh water
continually erodes the salt wedge. The amount of such erosion upstream
from a given point is always equal to the upstream discharge of salt
water at that point so long as the position of the wedge remains stable,
or until the fresh water discharge changes. If the fresh-water flow
decreases, the wedge slowly moves upstream; conversely, if the fresh-
water flow increases, the wedge is forced downstream to a new position.
As an example of the magnitude of intrusion in the lower Mississippi
River, salt water is found 135 miles upstream from the mouth (entrance
to Southwest Pass) following prolonged periods of low river discharge.
On the other hand, the wedge is pushed entirely out of the river during
periods of extremely high river discharge.

Salinity intrusion in Delaware and San Francisco Bays appears to
be in the form of a salinity front rather than a salt-water
wedge, and is illustrated by Fig. 2. Salinities decrease progressively
as distance from the ocean increases, and there is but little difference
between surface and bottom salinity at any given point in the estuary.
The salinity front advances and retreats with both tidal fluctuation
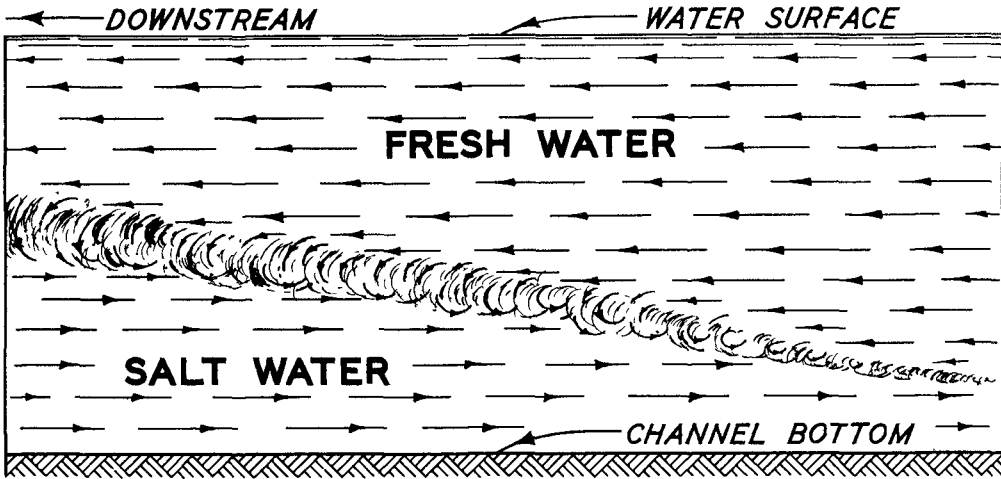
Fig. 1. This shows schematically the distribution of flow in an estuary having a well defined salt-water wedge (similar to lower Mississippi River). Flow in the wedge is always upstream, and flow in the fresh-water strata is always downstream. The thickness of the interfacial layer varies with the fresh-water discharge.
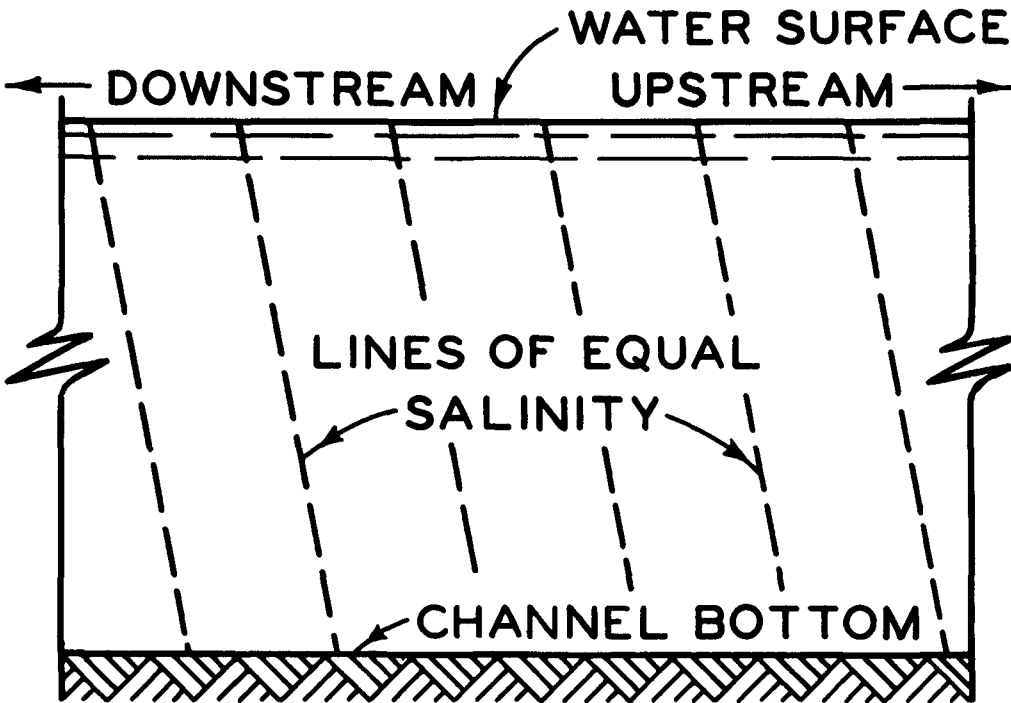
Fig. 2. This shows schematically the distribution of salinity in an estuary in which mixing caused by tidal turbulence does not permit formation of a well defined salt-water wedge (similar to San Francisco and Delaware Bays). The salt-water front advances and retreats with tidal action, but there is little difference in salinity from surface to bottom.

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and change in fresh-water discharge, but the relation between surface and bottom salinity at all points in the estuary does not change appreciably.

Salinity intrusion in the Savannah River, Georgia, and the Cooper River in South Carolina might be classified as falling between the two extremes described above and is illustrated by Fig. 3. There is a definite salt-water wedge in these latter rivers; however, the direction of flow within the wedge reverses with change in direction of the tidal currents, and the transition layer between the salt water and fresh water is much thicker than that found in the lower Mississippi River. The advance and retreat of the salt-water wedge with tidal fluctuation in the Savannah and Cooper Rivers covers appreciable distances, and the upstream penetration of the wedge varies with the volume of fresh water being discharged into the estuary. The shape of the wedge at high-water slack, or at maximum penetration because of tidal fluctuation, is somewhat different from that at low-water slack, the wedge having an appreciably steeper front and a lesser range of salinity distribution from surface to bottom than is found at low-water slack.

It is the opinion of the writer that the different types of salinity intrusion described above can be attributed to the physical and hydraulic characteristics of the estuaries involved. The lower Mississippi River occupies a channel which is relatively narrow as compared to depth, there is but little tidal range at the mouth of the river, and the fresh-water discharge is relatively large as compared to tidal discharge. On the other hand, the deep-water channels of Delaware and San Francisco bays are relatively narrow as compared to the total width of the estuaries (or the estuaries are relatively wide as compared to depth), the ranges of tide at the mouths of these estuaries are sufficient to produce reversals of flow with accompanying tidal currents of appreciable magnitude, and the fresh-water inflows are quite small as compared to tidal discharges. Physical and hydraulic conditions in these latter estuaries therefore contribute to the mixing of the salt and fresh waters, and since most of the fresh water enters the estuary at the upstream end, the salinity front type of intrusion exists rather than the wedge type. Conditions in the lower Mississippi River are such that but little mixing of the salt and fresh water takes place, and therefore salinity intrusion is in the form of a well-defined wedge. The other two rivers described, the Savannah and the Cooper, are similar in some respects to the lower Mississippi River and in other respects to Deleware and San Francisco Bays; therefore, the type of salinity intrusion in the Savannah and Cooper River have some of the characteristics of the former and some of the latter. The channels of the Savannah and Cooper Rivers are relatively narrow as compared to depth, the volumes of fresh water discharged into these estuaries are small as compared to tidal discharges, but the tidal ranges and accompanying tidal currents are sufficiently great to produce considerable mixing of the salt and fresh water. It is

SHAPE OF WEDGE AT HIGH TIDE

SHAPE OF WEDGE AT LOW TIDE
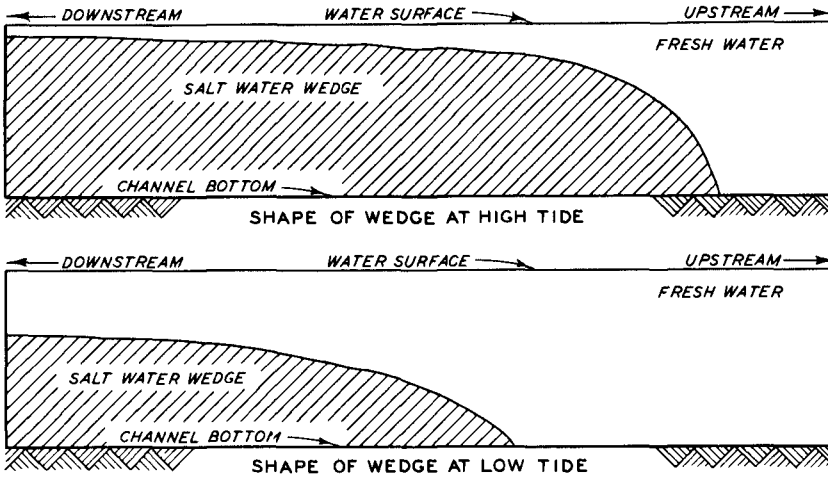
Fig. 3. This shows schematically the effect of tidal action on the salt-water wedge in an estuary having a well defined wedge but also having a sufficient tidal range to cause an appreoiable advance and retreat of the wedge with tidal action (similar to the Savannah and Cooper Rivers). The wedge moves back and forth for considerable distances with tidal action, and the upstream face of the wedge is much steeper at high-water slack than at low-water slack.
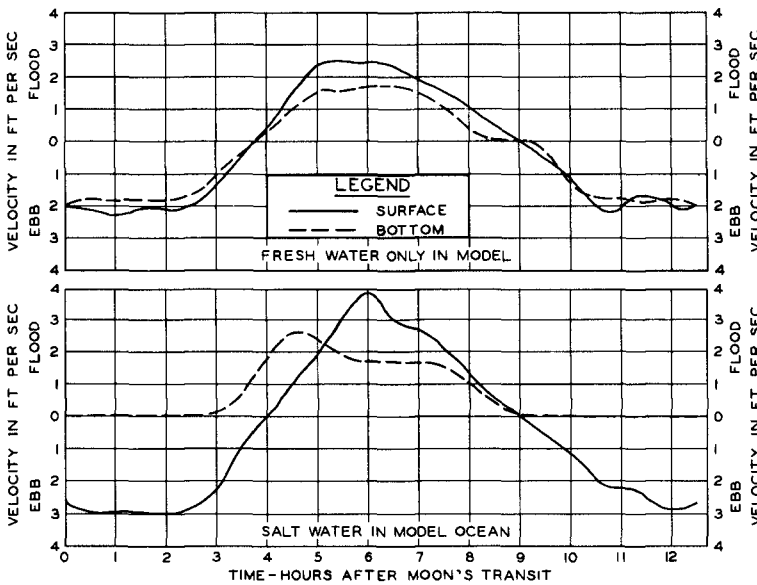
Fig. 4. This shows the effect on current velocities caused by using salt water in the ocean portion of the Savannah Harbor model. The upper plot consists of measurements made with only fresh water in the model, and the lower plot consists of measurements at the same station with salt water in the model ocean. The lower plot is similar to measurements made in the prototype for similar conditions of tide and fresh water discharge.

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therefore the opinion of the writer that while the greater density of the salt water as compared to that of the fresh is the basic cause of salinity intrusion in all open channels, the type of intrusion to be found in a given channel is so modified by local physical and hydraulic conditions that but little similarity exists between salinity intrusion in any two estuaries. Salinity intrusion in each estuary should be considered and studied as an individual problem, and the reasoning applied with success to one estuary will probably fail completely when applied to another.

CONTROL OF SALINITY INTRUSION

It has been pointed out that most salinity problems are unique in nature which would therefore require that each such problem be studied individually. For this reason it is believed that no set rules or methods for the solution of salinity problems can be developed. Experience and sound judgement on the part of the engineer concerned with a specific problem are required to determine the exact nature of the problem, what factors are of greatest significance, and what solutions are feasible and economical. Some of the attempts that have been made to remedy salt-water problems, and some of the proposals that have been made but have not yet been attempted, are described below.

LOCKS AND GUARD LOCKS

Locks are usually constructed to facilitate navigation by making it possible to transfer a ship from one level to another. However, in the transit of a ship from a body of salt water to a body of fresh water, or in the opposite direction, salt water passing through the locks during a ship transit can cause pollution of the fresh water body. It is sometimes desirable, therefore, to design and construct the lock in such a manner that its chambers may be flushed with fresh water before the gates separating the lock chambers and the fresh-water pool are opened. This process reduces the salinity of the water in the lock chamber, thus reducing the source of pollution to the body of fresh water. This method has been used with success in operation of the locks of the Lake Washington Ship Canal, Seattle, Washington,[5] and has been proposed for the solution of similar problems in other locations, notable of which is the proposed New Jersey Ship Canal.[6] The design of such locks usually incorporates a deep sump adjacent to the entrance of the lock into the fresh water pool, the purpose of which is to collect, by virtue of its greater density, the salt water that succeeds in passing through the locks in spite of all precautions. A siphon located in the bottom of this sump returns the salt water collected therein to the salt water pool below the locks. This method can be made quite effective if a sufficient quantity of fresh water for flushing the locks is available.

Guard locks are usually constructed to prevent free flow through an open channel, and for this reason they provide an excellent means for preventing the movement of density currents. The use of guard locks, however, is usually limited to waterways in which flow is principally tidal, since, in the case of a waterway having an appreciable fresh water run off, the lock would impede drainage of flood waters. Notable examples of existing guard locks are the Great Bridge Lock in the Atlantic Intracoastal Waterway about 12 miles south of Norfolk, Virginia,[7] and the Calcasieu and Mermentau Salt-water Guard Locks in the Gulf Intracoastal Waterway in southwestern Louisiana.[8]

BARRIER DAMS

A large number of barrier dams have been proposed for the control of salinity intrusion; however, only a few such structures have actually been built. Barrier dams in open channels are of two general types: (a) a dam which effects a partial closure of a channel, thus concentrating the fresh-water flow into a relatively small cross section and increasing its effectiveness in combating salinity intrusion; and (b) a dam which effects a complete closure of a channel, thus creating a fresh-water pool at an elevation equal to or slightly greater than the salt-water pool downstream from the basin. Either of these types may be supplemented by locks for navigation or auxiliary openings for passage of large fresh-water flows. Examples of the former are the barriers proposed for control of salt-water intrusion in the lower Mississippi River,[9] and examples of the latter are the Goolwa Barrage across the lower Goolwa River in Australia[10] and the barrier across the Santa Ynez River at Camp Cooke, California.[11]

Several types of movable barriers have been designed for the control of salinity intrusion, and a few of these have been constructed and operated. A movable tidal gate was constructed across the Miami River Canal, Miami, Florida, and operated for some time to prevent salt-water intrusion into the canal.[12] One-way gates to prevent intrusion of salt water at high tide, at the same time permitting drainage of fresh water at low tide, have been used successfully in a number of cases. Such gates are usually relatively small and are installed in highway and railroad culverts; however, fairly large gates of this type have been utilized also. An example of the larger gates of this type is that used in Skagit County, Washington, which controls drainage from and prevents salt water intrusion into an area of about 6000 acres of highly productive farm land.[13]

REDUCTION OF SHOALING IN TIDAL WATERWAYS

The problem of flocculation of suspended or dissolved solids by salt water is further complicated by the effects of density currents on

bottom velocities in estuaries. It has been pointed out that salt
water tends to move upstream because of its greater density, sometimes
underneath the fresh water and sometimes mixed with the fresh water in
various degrees, and the net effort of this upstream movement is that
bottom velocities in most estuaries are stronger during flood tide than
during ebb tide. Surface velocities, on the other hand, are usually
considerably stronger during ebb tide than during flood, since it appears
that most of the fresh water run-off is carried to sea in the upper
stratas, especially if the estuary is one in which little mixing of the
fresh and salt water occurs. The importance of such effect on the
vertical distribution of currents in an estuary is obvious, since it is
extremely difficult for a deposit of material, once formed on the river
bed, to be removed by natural forces and carried out to sea if the bottom
currents moving upstream are normally predominant over the downstream
currents. In fact, there is evidence that material flocculated in the
lower reaches of some estuaries is thence progressively moved upstream
by the predominant bottom currents to form shoals in the upper reaches
of the estuary.

The usual plans adopted to reduce shoaling in tidal estuaries
consist of proper channel alignment, the construction of training works
to obtain good flow conditions in the navigation channel, removal of
obstructions to the free run of the tide, elimination of channel bifur-
cation when possible to eliminate cross flows and eddy action, construc-
tion of jetties where required to concentrate flow, etc. It has also
been proposed on numerous occasions that the construction of reservoirs
in the fresh-water section upstream from the head of an estuary, the
revetment of banks to eliminate bank caving and scour, and the dredging
of sedimentation basins, either in the estuary proper or in the fresh
water section upstream, might be effective in reducing the suspended load
entering the estuary and therefore reduce the amount of solids available
for flocculation by salt water. Sedimentation basins located in the fresh
water section would probably be ineffective in reducing the suspended load,
however, and the cost of constructing reservoirs and bank revetment is
usually so great as to be prohibitive. Also, there is some evidence that
suspended material in appreciable quantities can pass through one or more
reservoirs without being deposited, only to be flocculated on coming into
contact with the salt water of an estuary and contributing to shoaling of
the navigation channels therein.

MODEL STUDIES OF SALINITY PROBLEMS

Hydraulic models have been used quite successfully during recent
years for studies of salinity intrusion problems in open channels and
in locks. Both theory and experience have shown that so long as gravity
is the controlling force in the formation and movement of density
currents, which is the case except in rare instances, models should be

designed in accordance with Froude's model laws of similitude, and the salinity scale, model to prototype, should be unity. The reliability of a salinity scale of unity for model studies of salinity intrusion in open channels was recently confirmed by the results of an exhaustive study conducted by the National Bureau of Standards under the direction of Dr. Garbis H. Keulegan.

O'Brien and Cherno, in 1934, published the results of a study to determine a model law for use in the design of hydraulic models for the study of salinity problems.[14] The design criterion arrived at by Messrs. O'Brien and Cherno was:

$$L_r = D_r^{2.5} S_r^{0.5}$$

(1)

This equation was based on the assumption that inertia and friction were the controlling forces in movement of salinity currents, gravity being considered only insofar as the initial velocity was concerned. This condition exists only if a relatively small volume of salt water is released into a large body of fresh water as, for example, a slug of salt water entering a fresh water pool as a result of lock operation, and is therefore limited in scope.

It is usually necessary to reproduce salinity currents in estuary models for two reasons. First, some industries usually draw cooling water directly from estuaries, and any change in salinity, caused by changes in channel dimensions or other modifications of the estuary, may increase salinities to or beyond the danger point. Second, as stated previously, the action of salinity or density currents is such that the vertical distribution of currents in any cross section traversed by a salt-water wedge is appreciably different than would be the case if all fresh water or all salt water was moving in the cross section. The upper portion of Fig. 4 shows a plot of surface and bottom current velocities obtained in a model of Savannah Harbor, Georgia, with only fresh water in the model. The lower portion shows a plot of velocities at the same points with salt water in the model ocean and fresh water being discharged into the upstream end of the model estuary. These plots reveal the great differences between surface and bottom velocities at an identical point for the two conditions mentioned. The lower plot approximates very closely the velocity distribution found in the proto-type for tidal and fresh-water flow conditions similar to those repro-duced in the model at the time these measurements were obtained.

It is obvious that the movement of sediment in Savannah Harbor is affected appreciably by density currents, especially in that reach of the harbor in which these measurements were made. The observations obtained with salt water in the model ocean show an appreciable flood or upstream velocity on the bottom in this critical reach, but the ebb or downstream velocity is zero. The net effect of this difference in

bottom velocity is that material deposited in the harbor because of
flocculation or other causes may be moved upstream to form shoals in
the navigation channel, but it can not be moved downstream and out to
sea except during times of extremely high fresh water flows. It
therefore follows that most of the material accumulated in critical
reaches by this peculiar flow phenomena must be removed by dredging to
maintain the desired channel depths.

In 1944-1945 a model study of salinity intrusion in the Calcasieu
River, which is located in southwest Louisiana, was made by the Waterways
Experiment Station, and this study may be used to illustrate the advantages
of using hydraulic models in such investigations.[15] The problem involved
was whether or not a proposed deepening of the Calcasieu River Ship
channel, between the Gulf of Mexico and the Port of Lake Charles, La.,
from 30 to 34 ft at MGL would further aggravate an already serious
salinity intrusion problem in the Calcasieu River and the Gulf Intra-
coastal Waterway east of the Calcasieu River. Water drawn from these
streams, especially from the Intracoastal Waterway east of the Calcasieu
River, is used for irrigating large areas used for rice cultivation.

One of the initial steps in planning the model study was the
obtaining of sufficient and reliable hydraulic and salinity data upon
which the adjustment of the model could be based. Tidal heights were
obtained by automatic tide-recording gages in the Calcasieu River,
Calcasieu Lake, and in the Intracoastal Waterway east and west of the
Calcasieu River. Measurements of current velocity and salinity were made
at six selected stations in the area to be reproduced. Fresh water
discharges in the Calcasieu River and all tributaries were made daily
for the duration of all tidal, velocity, and salinity observations. These
data provided an accurate basis for adjustment of hydraulic phenomena and
verification of salinity intrusion for the then-existing 30-ft channel.

A comprehensive study of salinity intrusion in the Calcasieu River
area by means of a hydraulic model required that the model be so designed
and constructed that both tidal and salinity currents be reproduced
accurately throughout the problem area. Therefore, the model reproduced
a portion of the Gulf of Mexico, Calcasieu Pass and Calcasieu Lake, 10
miles of the Lake Charles Deep Water Channel to the west of the Calcasieu,
10 miles of the Gulf Intracoastal Waterway to the east of the Calcasieu,
the Calcasieu River in its natural state to a point about 5 miles above
Lake Charles, and the remaining tidal portions of the Calcasieu River,
Houston River, and English Bayou above this point in the form of a
labyrinth. Fig. 5.

The model was of the fixed-bed type, all channel and overbank areas
being molded in concrete. It was constructed to linear scale ratios, model
to prototype, of 1:1000 horizontally and 1:50 vertically. The salinity

scale used in the model was 1:1. Observed prototype tides were
reproduced in the simulated Gulf of Mexico by means of an electro-
mechanical tide reproducing apparatus, and the roughness values of the
model bed and channels were so adjusted that the observed rise and fall
of the tides, and the resulting strength and directions of tidal
currents, were reproduced throughout the model. The fresh-water inflows,
representing the fresh-water discharges of the Calcasieu and Houston
Rivers, were introduced into the upstream end of the model by means of
Van Leer weirs.

The water in the model gulf was maintained at the observed proto-
type salinity by the addition of common salt. The salt was added to the
water-supply sump in the required quantities, and was dissolved by a
circulating pump. Since the water which produced the rise and fall of
the tide in the model circulated through this sump, the water in the
model gulf was maintained at its correct salinity at all times.

Operation of the model for the verification test, and for tests of
the various channel conditions for which information was desired, was
begun with the river system filled with fresh water and the gulf filled
with salt water, the two bodies of water being separated by a movable
block located at the lower end of Calcasieu Lake. The tide control
mechanism was then started and the block was removed, the fresh-water
inflow weirs having been adjusted to reproduce the desired inflow for
that particular test.

It is obvious that conditions in any tidal stream in nature repre-
sent an adjustment between the forces of fresh-water flow and those of
tidal flow. It can be readily understood, therefore, that these forces
in the model must be allowed to adjust themselves before a state of
stabilization is reached. In the model, where factors affecting the
adjustment between these forces are controlled, as they are not in the
prototype, such a state of stabilization is characterized by the
repetition of the action of the salt-water wedge during successive cycles
of operation. When the position of the wedge becomes stable, or the
advance and retreat of the wedge with tidal action is constant, the
wedge may then be considered to have penetrated as far into the river as
it will for the conditions reproduced.

For verification of the Calcasieu River model, the conditions of
tidal action, fresh-water inflow, and gulf salinity were reproduced in
the model in accordance with prototype data obtained during the field
survey of the prototype previously mentioned. After the model was
operated through the stabilization period, salinity samples were obtained
at all stations and depths corresponding to the stations and depths at
which prototype measurements were obtained. The results obtained from
the model were then compared to data obtained in the prototype for similar

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TABLE I

SALINITY MEASUREMENTS -- VERIFICATION TEST

Salinity values are expressed in parts per million
and represent average of all samples obtained during one tidal cycle

Salinity Station	Depth Sample Taken	Average Salinity for one Tidal Cycle	
		Model	Prototype
1	0.0	2,360	3,280
1	-15.0	5,625	5,290
1	-30.0	14,790	12,670
2	0.0	500	430
2	-14.0	500	430
2	-28.0	12,340	9,740
3	0.0	400	330
3	-16.0	500	418
3	-31.0	14,520	7,150
4	0.0	75	73
4	-15.0	80	74
4	-30.0	85	76
5	0.0	400	320
5	-6.0	400	319
5	-12.0	400	322
6	0.0	700	657
6	-14.0	700	669
6	-27.0	2,000	1,968
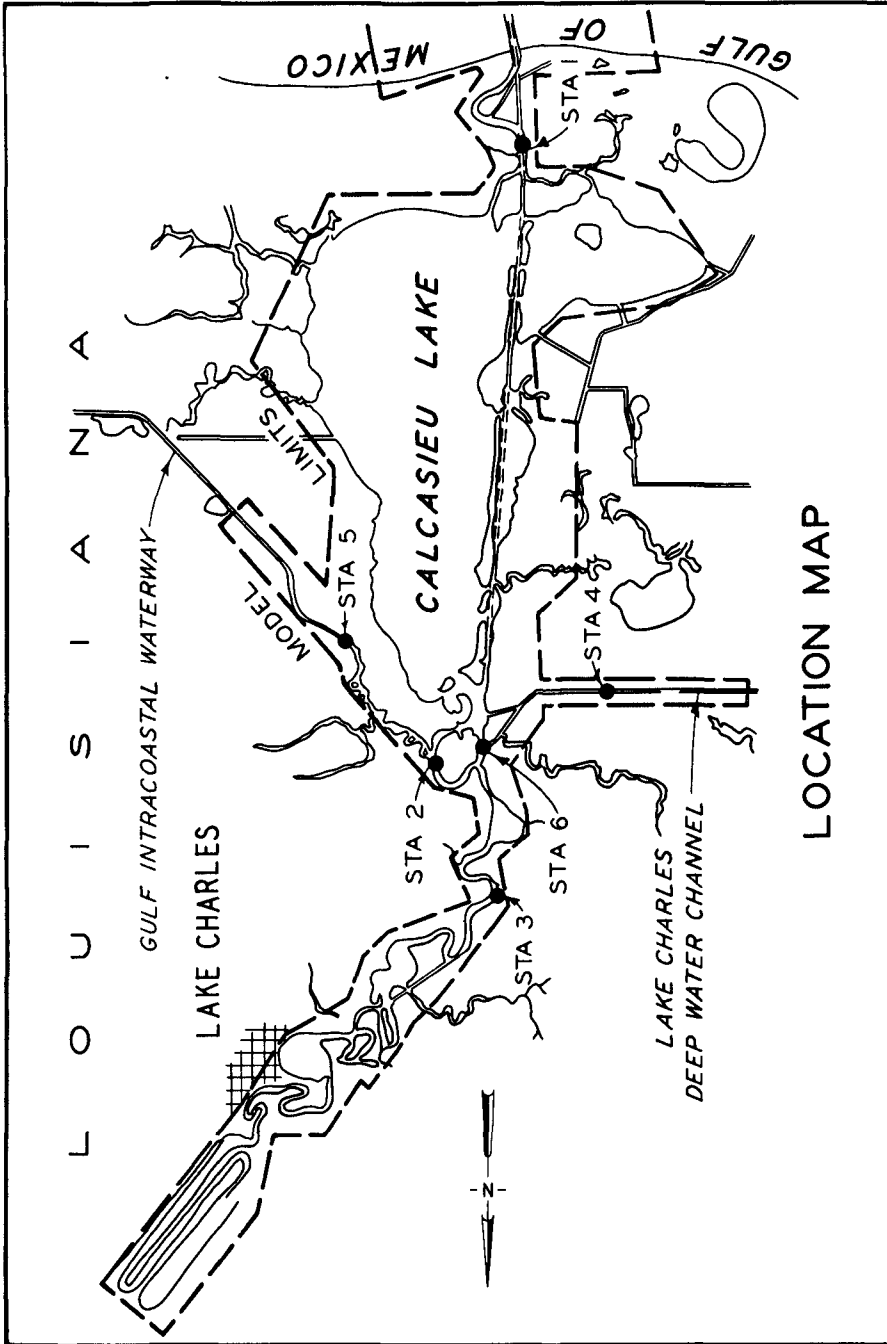
Fig. 5. This shows the location of the Calcasieu River, Louisiana, and connecting water-
ways, the limits of the Calcasieu River model, and the location of salinity stations at
which comparable model and prototype salinity verification data were obtained.

conditions (table 1), and it was found that salinities in the model checked those obtained in the prototype with a remarkable degree of accuracy. The verification test was repeated several times to insure that identical and consistent results would be obtained.

After the verification of the model had been established, extensive salinity measurements were obtained for a series of fresh water river discharges and a series of eastward and westward flows in the Gulf Intracoastal Waterway. These various test conditions were selected with a view toward obtaining salinity data for any combination of flow which might be expected to occur in the prototype. Sufficient data were obtained during each test to serve as a basis for comparison of the results of later tests made after deepening the project channel. After all tests of the existing channel had been completed, the model channel was deepened to the new project and the series of tests was repeated. It was found that deepening the channel had very little effect on salinities in the Calcasieu River or in the Gulf Intracoastal Waterway east of the Calcasieu. Slight local differences were noted, usually occurring at middepth of the channel; nowever, there was no apparent overall change in salinities.

Following a thorough analysis of the results of all model tests, it was concluded that the proposed channel deepening would have no adverse effects on salinity intrusion in the Calcasieu River and the Intracoastal Waterway east of the Calcasieu. Since salinity intrusion for the 30-ft channel caused a serious problem at certain times, it was further concluded that a guard lock in the waterway should be constructed to alleviate this condition. A guard lock was constructed in the Intracoastal Waterway near Black Bayou, and it is the understanding of the writer that this lock has been very successful in preventing contamination of irrigation water therein by the intrusion of salt water from the Calcasieu River.

CONCLUDING REMARKS

This paper has covered only the general aspects of salinity problems and possible solutions, and was prepared to stimulate the thinking of other engineers in this respect rather than to show how salinity problems should be solved. The people of this country, especially the engineers, are constantly devoting more and more thought to water conservation and proper use of water, and it is the writer's opinion that salinity problems will receive more attention and study as inland sources of water are gradually developed to the fullest extent possible. The day may soon arrive when we, like the Dutch, will find it necessary or profitable to reclaim marginal land from the sea by the exclusion of salt water, and in doing so it is inevitable that new and startling engineering solutions will be developed.

SALINITY PROBLEMS

REFERENCES

1. U.S., National Resources Planning Board: Regional Planning, Part X, Pecos River Joint Investigation in the Pecos River Basin in New Mexico and Texas; Summary, Analysis, Findings, 1942.

2. Louisiana, Department of Conservation: Calcasieu River Survey, 1942.

3. Matthes, G.H.: Flocculation of River Clays Through Contact with Salt Water. New York, N.Y., December 1948.

4. Hynes, V.M.: Salt Content of Wells in Massachusetts Affected by the Tidal Wave and Hurricane of 1938. (New England Water Works Association Journal, 56: 355-360, September 1942.)

5. Barden,W.J., and Sargent, A.W.: Lake Washington Ship Canal (Trans., A.S.C.E., 92: 1001-40.)

6. Waterways Experiment Station: Model Studies of Water Requirements and Salt Water Intrusion, Intracoastal Waterway, New York Bay-Delaware River Section. Final Report, 1946.

7. Young, G.R.: The Great Bridge Lock. (Military Engineer, Vol. xxiv, November-December 1932, 138: 551-557.)

8. U.S., Department of the Army, Corps of Engineers: Definite Project Report, Mermentau River Basin, Mermentau River, Louisiana, Flood Control, Irrigation, and Navigation, 1948.

9. U.S., War Department, Corps of Engineers: Salinity, Mississippi River at and Below New Orleans, June 1942.

10. Lawrie, E.R.P.: Construction of the Goolwa Barrage. (Australian Engineers Journal, August 1940, 12: 221-231.)

11. Beardslee, C.G.: Salt Water Barrier at Cooke. (Western Construction News, 17: 53-55, February 1942.)

12. Wailes – Dove – Hermiston Corporation: Checking the Tide Waters of the Everglades. Westfield, 1944. (ITS Bitumastic Bulletin No. 30.)

13. Muccus, F.K.: Skagit Bay Tide Gate Repaired. (Western Construction News, 17: 100-102, August 1945.)

14. O'Brien, M.P., and Cherno, J.: Model Law for Motion of Salt Water through Fresh. (Trans., A.S.C.E., 99: 567-609, 1934)

15. Waterways Experiment Station: Salt-Water Intrusion in the Calcasieu River, Louisiana, and Connecting Waterways; Model Investigation. Final Report, 1949.

16. Barksdale, H.C.: Ground Water Problems in New Jersey. (Journal American Water Works Association, 34: 563-68, June 1945.)

17. Brenneke, A.M.: Control of Salt Water Intrusion in Texas. (Journal, American Water Works Association, 37: 579-84, June 1945.)

18. California, Department of Public Works, Division of Water Resources: Economic Aspects of a Salt Water Barrier below Confluence of Sacramento and San Joaquin Rivers, 1932. (ITS Bulletin No. 28.)

19. California, Department of Public Works, Division of Water Resources: Report on Salt Water Barrier below Confluence of Sacramento and San Joaquin Rivers, California. Sacramento, 1949. (ITS Bulletin No. 22.)

20. California, Department of Public Works, Division of Water Resources: Variation and Control of Salinity in Sacramento-San Joaquin Delta and upper San Francisco Bay,1931, Sacramento, 1932. (ITS Bulletin No. 27.)

21. California, Department of Public Works, Division of Water Resources: Proposed Major Development on American River; Analysis of its utility in the coordinated plan for the development of the water resources of California, 1930. (ITS Bulletin No. 24.)

22. Eldridge, E.P.: Industrial Waste Treatment Practice, 1942.

23. Fries, F.E., and Viney, A.G.: Hydraulic Model Test of Lake Washington Ship Canal; Thesis, 1933. (Engineering News Record, June 1933.)

24. Kelley, W.P.: Permissible Composition and Contraction of Irrigation Water. (Transactions, A.S.C.E., 1941, 106: 849-861.)

25. Parker, G.G.: Salt Water Encroachment in Southern Florida. (Journal, American Water Works Association, 37: 526-542, June 1945.)

26. Pennsylvania, State Department of Health, Bureau of Engineering: Final Report to the Sanitary Water Board by the Bureau of Engineering on the Salinity Survey of the Delaware River. Harrisburg, May 1935, 2 Vol.

27. Poland, J.F.: Saline Contamination of Coastal Water in Southern Californ (Western City, 19: 46, 48, 50, October 1943.)

28. Riddel, J.O.: Excluding Salt Water from Island Wells. (Civil Engineerin 3: 383-385, July 1933.)

29. Scofield, C.S.: Measuring the Salinity of Irrigation waters and of Soil Solutions with the Wheatstone bridge. July 1932. (U.S. Department of Agriculture, Circular No. 232).

30. Service, J.H.: Measurement of Salinity of Sea water, (U.S. Coast Geodetic Survey, Special Publications No. 147, 1928.)

31. U.S., Works Progress Administration: Salinity of the Connecticut River October 1934-September 1937. Hartford, Conn., October 1938-June 1939 3 Vol.

32. Waterways Experiment Station: Model Study of Salt Water Intrusion, Lower Mississippi River. Preliminary Report, May 1942.

33. Waterways Experiment Station: Plans for the Improvement of the St. Johns River, Jacksonville to the Ocean; Model Investigation. Final Report, 1947.

34. Waterways Experiment Station: Plans for Improvement of Navigation Conditions and Elimination of Shoaling in Savannah Harbor, Georgia, and Connecting Waterways; Model Investigation, Final Report, 1949.

35. Wood, A.B.: Saline Conditions of the Mississippi River, New Orleans, 1937.

Chapter 8

ACTION OF MARINE BORERS AND
PROTECTIVE MEASURES AGAINST ATTACK

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United States Navy

SUMMARY

A major concern to engineers engaged in the design of timber harbor structures is protection against marine borers. These pests can severely damage these structures in a relatively short time. Attack is concentrated on submerged timbers in the area between the mudline and the water surface. The intensity of attack is dependent on a number of environmental conditions. The most destructive and widely distributed borers are the Teredinidae and the Limnoria. Some forms of borers exist in all oceans. This paper describes the manner in which the borers destroy timber. It summarizes information gathered by various investigators on the conditions which have bearing on the presence of borers and the factors governing rate of destruction. Several methods of protecting timber structures from infestation are described and the costs compared.

INTRODUCTION

Marine borers have been defined as "marine invertebrates which drill into and consequently damage timber and other materials in salt water" - (U. S. Navy, 1950a). Marine borers are responsible for many millions of dollars of damage to harbor structures annually. These damages amounted to $100,000,000 in 1949. The Bureau of Yards and Docks, U. S. Navy, has reported cases where green piling up to 16 in. in diameter was severed in six months and treated timber was replaced in less than two years. An article by Long (1951) relates the story of a crew in a fishing boat which tied up to a pier in San Francisco Bay during lunch hour and returned after lunch to find no sign of either the boat or the pier. Later evidence disclosed that a wave caused by a passing ship had collapsed the pier and both boat and pier floated away on the tide. Both were found the next day, still lashed together. Another example of the destructive effect of marine borers relates to the Navy piers on Guantanamo Bay, Cuba. The piers, constructed during World War II, were of creosoted timber piling--(U.S. Navy, 1951a). The rate of destruction of one of these structures was obtained from an account of several surveys. Originally the minimum allowable pile diameter was 12 inches. A survey in the spring of 1949 showed that the minimum cross-section of the 13 outboard bents
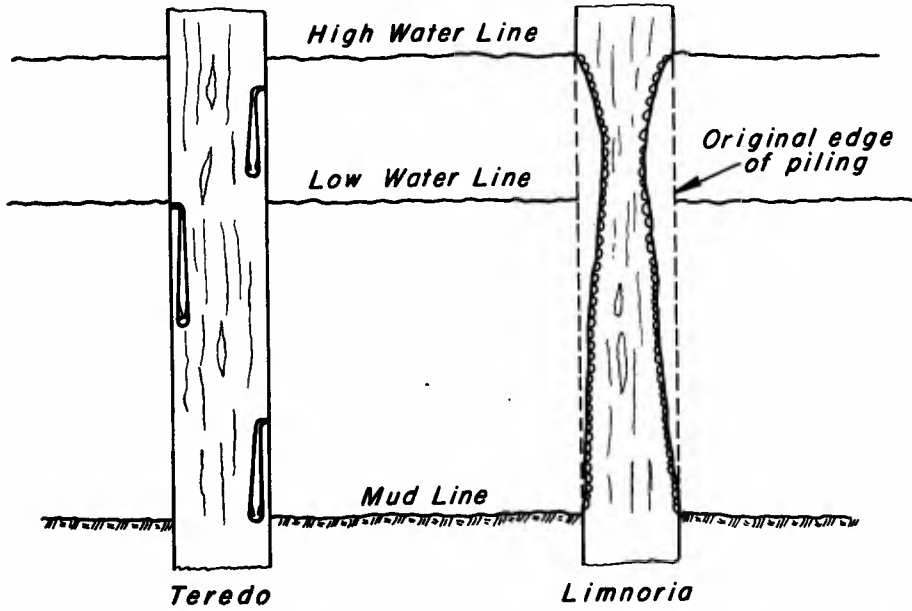
Fig. 1
Idealized cross-sectional sketches of piling showing typical
methods of marine borer attack.
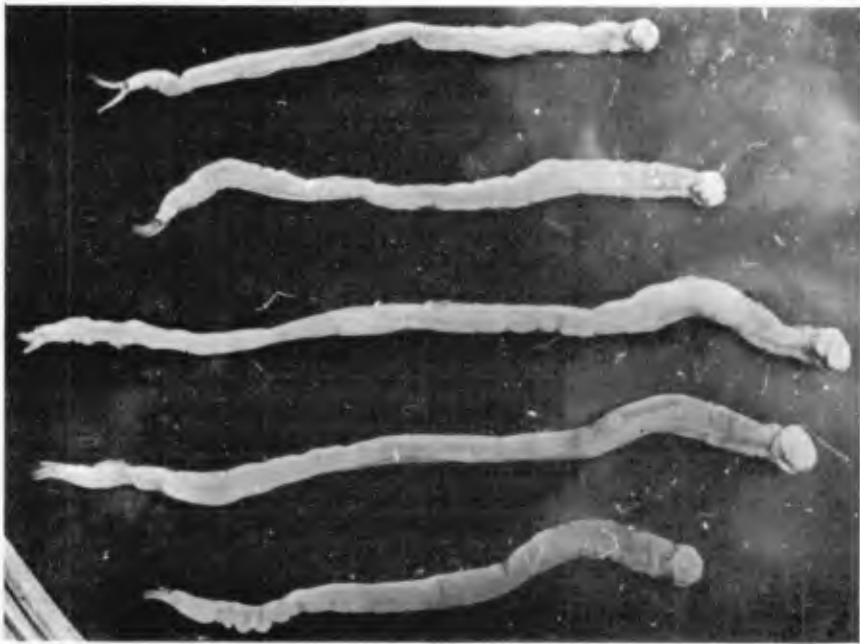
Fig. 2
Teredo Navalis.

average only 37 percent of the original section. By November 1949 these same bents showed only 17 percent of the original section. By February 1950, only 9 percent remained and observers concluded that because of honey-combing in the interiors of the piling probably only 50 percent of this 9 percent could be considered effective.

In the past few years the increased cost of the production and protection of timber has resulted in a considerable decrease in the use of wood for harbor structures. Nevertheless, there are many situations where the use of concrete is not justifiable, particularly in structures of rather temporary nature, as the case may be in defense installations. Therefore, continued study of the action of marine borers is certainly justified.

PRINCIPAL TYPES OF BORERS

Virtually all investigators agree that marine borers most damaging to harbor structures are (1) mollusks, of which the Teredo and its immediate relatives are the most destructive and widely distributed, and (2) crustaceans, of which the Limnoria is the most destructive and widely distributed. Another mollusk of importance is the Martesia; other crustaceans are the Chelura and Sphaeroma.

MANNER OF ATTACK

Borers of the molluskan and the crustacean groups attack the wood by different methods. Mollusks enter through very small holes bored when young and grow to size inside. This destruction can only be detected by cutting the wood or by careful inspection of the surface. The crustaceans, on the other hand, destroy the surface of the wood; thus attack is readily visible from the surface. The manner of attack of both groups is sketched on Fig. 1.

The Teredo usually enters the wood as larvae by boring holes as small as 0.008 in. in diameter which increase to about 0.03 in. -- (Chellis, 1948). After a larva enters, it starts to grow. Normally the adult size is of the order of 3 to 6 in. in length with a diameter approximately that of a lead pencil; but occasional specimens have been reported in excess of 3 ft. in length, boring holes 3/4 in. in diameter or larger. The Teredo navalis is shown on Fig. 2. The head is equipped with two small shells similar to clam shells. The burrowing is performed by the back and forth motion of these shells. The tail part consists of two pallets, attached to a muscular collar, which in turn is attached to the wall of the burrow over the entrance. The shells and body grow in diameter as the burrow is made longer. The animal can seal the burrow entrance at will with the pallets and is thus able to withstand for some time unfavorable external conditions. With favorable conditions a single animal can bore as much as 14 in. per year--(U. S. Navy, 1950a). The average rate for the larger animals in San Francisco Bay is shown on Fig. 3. The life span of a Teredo does not often exceed one year.

The Limnoria is a relatively small lobster-like animal 1/8 to 1/4 in. in length and one-third to one-half as broad; about the size of a grain of rice. A Limnoria is shown on Fig. 4. It has horny boring mandibles, two sets of antennae, and seven sets of legs with sharp hooked claws. The mandibles are large denticulated organs which cut into the wood and crush it. The body is firmly held in position by the legs and claw-like feet. The Burrows are .025 to .50 in. in diameter. A large number of Limnoria reduce the surface of the wood to a network of burrows. To the casual observer the surface has the appearance of a sponge. Limnoria are the primary cause of the char-acteristic hour-glass shape of piling shown in Fig. 1. In contrast with the Teredo which must locate wood on which to settle while in the larval stage, Limnoria are able to swim short distances throughout their life and therefore can change habitat at any time. It is repor-ted that they can burrow into soft woods such as pine to a depth of about one inch per year--(Chellis, 1948).

AREA OF ATTACK

The main area of attack of the borers extends from the mudline to the high water line. Previously many engineers thought that Limnoria attack was greatest at the surface and Teredo greatest at the mudline. However, one of the earliest results of the Navy research program was to show that such is not necessarily the case. In Florida, attack by both Teredo and Limnoria was on the average three times more intense near the mudline than at the surface, although at two locations in water of high salinity the greatest attack was at the surface. The observers concluded that buoyancy was probably responsible; the area of attack being higher in denser water. The San Francisco Bay Marine Piling Committee found that borer action is greater in deep than in shallow water; Teredo attack being greatest towards the mudline and Limnoria attack greatest between tide levels--(Hill and Kofoid, 1927).

ENVIRONMENTAL INFLUENCES

Salinity, temperature, abundance of food, current action, pol-lution, dissolved oxygen, hydrogen-ion concentration, and the amount of dissolved hydrogen sulfide all may have some bearing on the presence and the amount of activity of marine borers. Investigators have found that the first three are the most important. Various attempts have been made to relate intensity of attack with these factors. Geographic ranges have been difficult to establish since places which are rela-tively safe from attack at one time may suffer severe attack at another. A report on marine borer activity in 56 important harbors was compiled by the U. S. Navy--(1951b). Particular species sometimes are pre-dominant in an area. Absense of borers in an area is no assurance that future attacks will not occur. Also borer populations may change rapidly over a period of a few years. For example, along the North Atlantic coast of the United States from 1939 to 1941, the borer pop-ulation was estimated at 100 times what it was a few years earlier--(Chellis, 1948).

Salinity--Changes in salinity as small as 10 parts per 1000 may have a marked effect on borer activity. Ocean salinity is about 30 to 35 parts of salt per 1000. Teredo navalis activity decreases rapidly in salinities of less than 9 parts per 1000 and the threshold of lethal salinity has been placed at about 5 parts per 1000. Limnoria seem to require higher salinities; about 30 parts per 1000 for full activity; there is a decided retardation in salinities of 12 to 16 parts per 1000; and 6.5 to 10 parts per 1000 appears to be lethal.

Temperature--The breeding period of the Teredo is influenced to a great degree by the temperature of the water. For the free swiming larvae to survive the water must be warm. Warm water is also a stimulus for the parent to expel the egg or larva as the case may be. Therefore, it follows that the breeding season is much longer in the southern coasts of the United States than in the north. In general, the breeding season and the period of heaviest attack for Teredo navalis include the summer and autumn months of the year. Temperature has very little effect on Teredo after it has embedded itself in the wood and it will continue to attack unless the temperature falls to slightly above freezing, when it lies dormant.

Investigators have found that Limnoria is not affected to the same extent as the Teredo by temperature. Limnoria is constantly present in regions as far north as Kodiak, Alaska. In San Francisco Bay breeding occurs when the temperature is as low as $46^{\circ}F$. At Kodiak the water temperatures are considerably below 46 degrees Fahrenheit. As for Teredo, attack will continue until water temperature decreases to a point just above freezing.

Food--The Teredo gets its main supply of food from plankton. It requires proteins for growth and receives this diet from plankton; however, energy for the boring process is provided principally by the carbohydrates of the wood. This energy can be supplied from the protein material of the plankton but much less efficiently since nitrogenous products must be eliminated in the process-(Hill and Kofoid, 1927). More recent studies--(U. S. Navy, 1950a) show that this early conclusion was essentially correct. In laboratory studies where the Teredo were left in the wood but deprived of plankton, they showed very little change after 7 days. However, when removed from the wood and placed in a plankton-rich environment, only one animal in an extensive series of tests survived more than 7 days. Chemical tests in this series showed that the wood was used to sustain carbohydrate metabolism. The important fact is that the Teredo cannot live on plankton alone. Consequently, the impregnation of wood is a rational method of introducing toxins into the animal's system. In contrast, the Limnoria derives all its food from the timber it attacks and does not require plankton.

Pollution and Sewage--Heavy pollution in many cases seems to prevent action by marine borers. Yet numerous wooden sewer outlets have been completely destroyed by Teredo. Investigations by the Marine Laboratory, University of Miami, indicate that domestic sewage is
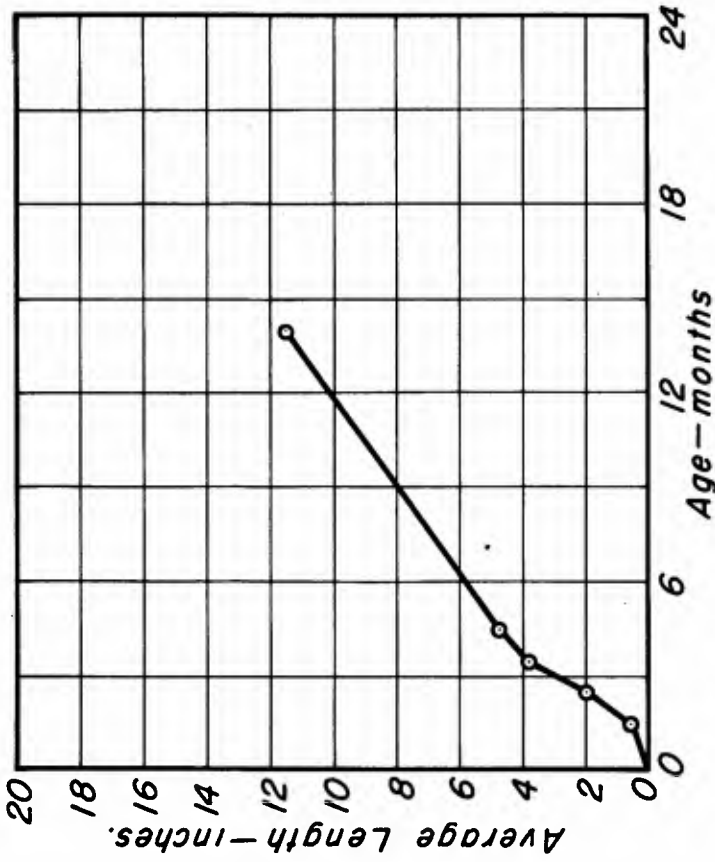
Fig. 4.
Limnoria.

Fig. 3
Rate of boring of Teredo Navalis in San Francisco Bay.

Average Length—inches.
Age—months
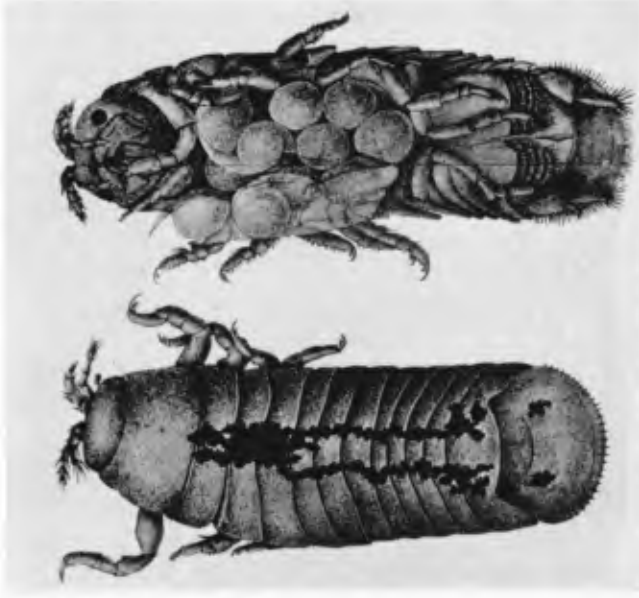
beneficial rather than harmful to marine borers. It is the opinion of many of the investigators that it is industrial waste rather than domestic sewage that provides conditions unfavorable for borers.

Dissolved Gases and Hydrogen-ion Concentration--Marine borers seem to be adaptable to wide variation in the amount of dissolved oxygen and dissolved hydrogen sulfide, and in the hydrogen-ion concentration of the sea water. The amount of dissolved oxygen in normal sea water is of the order of 5.5 cc per liter. The San Francisco Bay Marine Piling Committee found that the average hydrogen sulfide content in San Francisco Bay was .13 cc per liter and the maximum was .42 cc per liter. The hydrogen-ion concentration is a measure of acidity or alkalinity. Normal sea water is slightly on the alkaline side and has a pH value of 7.5 to 8.5. At the present time sufficient information is not available to establish specific lethal values for these factors.

Current Action--Experiments conducted by the Marine Laboratory, University of Miami, to determine the effect of current velocity upon the attachment and growth of several borers, including Teredo and Limnoria, indicated that currents of the order of two knots seemed to preclude attack in Florida waters. Fig. 5, obtained from data collected by the University of Miami, indicates the effect of current action on intensity of borer attack.

PROTECTIVE MEASURES

There are many schemes for protecting piles from marine borer attack. The choice of method depends to a great extent upon the availability of protective materials, their cost, and the economic life of the structure to be protected. Care in application of protective measures is vital. Minute openings offer entrances for many of the most destructive types of borers. This paper describes three methods which have been in recent use in the United States; these are (1) creosote pressure treatment, (2) gunnite jackets, and (3) precast concrete jackets.

CREOSOTE PRESSURE TREATMENT

Inventors have been busy for years endeavoring to find ways and means of protecting wood against decay and marine borers. It was not until 1838, when John Bethel patented the "Full Cell Process", that real progress in wood preservation in this country began. This process effects a deep penetration of the preservative; the depth being dependent upon a number of factors, chief of which are the kind of wood, character of growth, its condition as to seasoning, the method of treatment, and the preservative used. Non-pressure processes are not recommended for piles to be used in salt water as the depth of penetration, even under the most favorable conditions, is extremely small. Either green or seasoned timber can be treated by the pressure process. Green timber is often seasoned by means of live steam before treatment.

Fig. 6
Cross-section of gunite jacketed pile.

2 x 2 mesh

2 in. thick Gunite

Teredo

Limnoria

Average computed Velocity — knots
(From data obtained June 1 – 27, 1949, –
Miami, Florida.)

No. of borings per 100 sq. in.
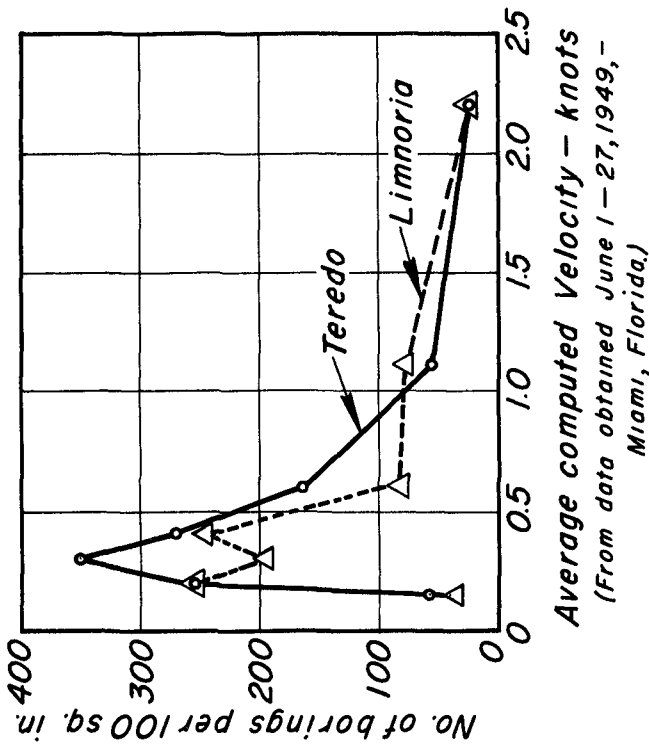
Fig. 5
An indication of intensity of borer attack vs.
water current velocity.

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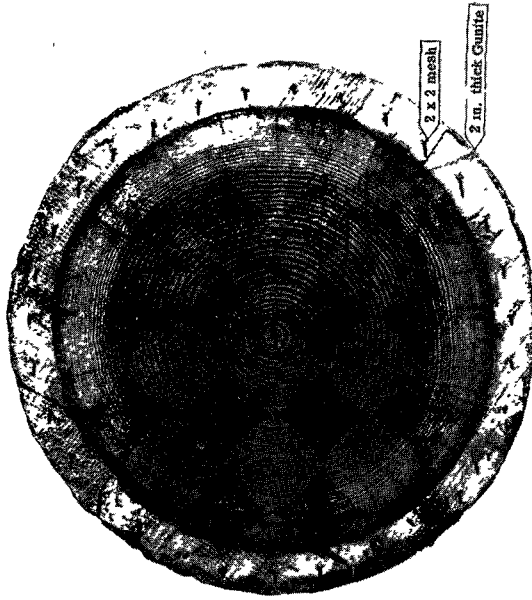
Of the available preservatives, creosote remains the most dependable means of reducing marine borer attack. Creosote is produced by high temperature carbonization of bituminous coal; to increase fluidity, coal tar or petroleum are often added. These admixtures are not toxic to marine borers, therefore their use is not recommended where borers are present.

Although the amount of creosote and the depth of penetration vary considerably, depending on the character and the density of the timber, the American Wood Preservers Association specify the following for Southern Pine and Pacific Coast Douglas Fir subject to borer attack.

	Retention lb per cu ft (minimum)	Penetration (inches)
Southern Pine	16 to 24	4
Douglas Fir	12	3/4, 7/8, and 1 for 12, 14, and 16 lb retentions for full cell process.

Although the AWPA allows the use of either the full-cell or empty-cell processes, the full-cell process is preferred, because it not only coats the wood cells but fills the void spaces between them; the empty cell process only coats the cells. By filling the voids the full cell process assures the complete coating of the cells and provides a greater reservoir against leaching.

It will be noted that the protective coating for Douglas Fir is quite thin, hence extreme care is required against damage to the surface by abrasion. Whenever possible all bolt holes and notches should be cut prior to treatment.

Creosote treatment does not result in complete resistance to borer attack but the progress in creosoted piles is at a much slower rate. In Carribean waters an average life of 8 to 10 years is all that can be expected; in San Pedro Harbor from 20 to 30 years; in San Francisco Bay, 15 to 25 years on the San Francisco side and 20 to 30 years elsewhere.

GUNITE JACKETS

Gunite or pressure-applied concrete has been used successfully on wood piles for some years. The gunite jacket is applied before driving, over that portion of the pile which is exposed to attack by marine borers. The jacket protects the pile from decay, rust, erosion, and fire. It is extended a few feet below the anticipated depth of scour. The average thickness of jackets is 2 inches; a cross section

Fig. 7. Gunite being placed on wood pile.
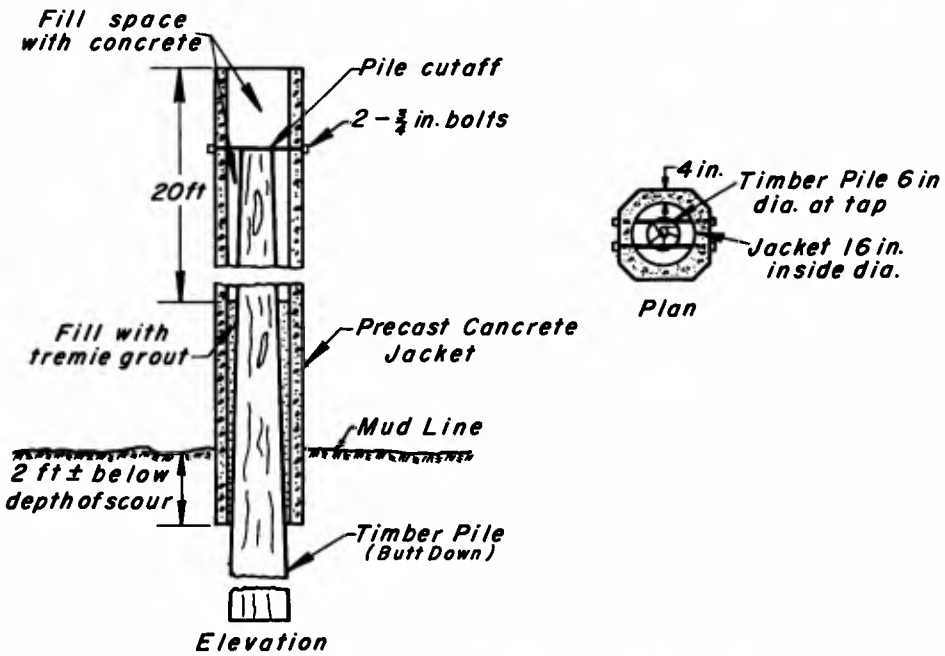
Fig. 8. Details of precast concrete jacket.

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is shown on Fig. 6. The steel reinforcement usually consists of galvanized electrically-welded fabric of No. 12 gauge (.1055 in. diameter), 2 x 2 in. mesh. The mesh is placed approximately one inch from the surface of the pile. In order to prevent injury to the protective jacket, the top 3 to 5 ft at the butt end is left uncoated until after driving. Fig. 7 shows gunite being placed on a wood pile. On the average, it takes approximately one week to apply the jacket and. cure it.

PRECAST CONCRETE JACKETS

There are several forms of precast concrete jackets; they differ only in detail. A precast concrete jacket developed by the Board of State Harbor Commissioners for the Port of San Francisco is described herein. The jacket consists of a circular inner face and a square outer face with beveled corners. The inside diameter of the jacket is 16 in. and the thickness of concrete 4 in. The green wood pile is driven butt down and cut off square at a predetermined elevation-- usually about two feet below the elevation of the deck. The jacket is placed over the pile and lowered or driven into the bottom material. It is supported on the timber pile by two 3/4 in. bolts which pass through the jacket (see Fig. 8). The precast jacket is extended a few feet below the anticipated depth of scour. The space between the pile and the jacket is sealed by grouting through a tremie pipe to 20 feet below the top of the jacket. The remaining space between the jacket and the pile is dewatered and filled with concrete.

COSTS

Comparative costs for creosoting, guniting, and precast jackets will vary depending on local sources of material, labor, contractors' equipment, and experience. Assuming that materials, labor, and equipment are readily available, the cost will depend largely on the length of the pile and the portion which requires protection. Because of the numerous factors involved, each job must be estimated in detail on its own merits. Since the increase in cost of timber in recent years has been disproportinate to the increase in cost of concrete, and because of the scarcity of long timber piles, there are many situations in which the concrete pile compares favorably in cost with protected timber piles.

Unit Costs--In San Francisco creosoted Douglas Fir piles 100 ft in length cost about $2.05 per lineal foot exclusive of driving. Likewise, untreated fir piling of the same length are in the order of $0.75 to $0.90 per foot. Gunite jackets approximate $2.25 per lineal foot. Precast concrete jackets in San Francisco (Fig. 8) were estimated at approximately $199,000 for 58,661 feet of jacket resulting in a unit cost of $3.40 per foot.

Sample Cost Comparison for 100 Foot Pile--Assuming that 35 feet of pile requires protection against marine borers, the following are

estimated costs for furnishing and driving 100 ft piles.

Creosote Pile	– – Furnishing (100 x 2.05)	$205
	Driving	15
	Total	$220

'Gunite Pile	– – – Furnishing Green Pile (100 x 0.80)	$ 80
	35 ft Jacket (35 x 2.25)	79
	Driving	15
	Total	$174

Precast Jacket	– Furnishing Green Pile (100 x .80)	$ 80
	Precast Jacket (35 x 3.40)	119
	Grouting and Filling Jacket	24
	Driving and Placing Jacket	18
	Total	$241

Reinforced Concrete Pile – Furnishing and Driving (100x4.20) $420.

RESEARCH

There are many investigators currently conducting research on marine borer activity. The two recent conferences on this subject; the Meeting to Discuss Marine Borer Situation (1949) at San Francisco, and the Marine Borer Conference (1951) at Port Hueneme, California are an indication of the interest in this field. The question of wood preservation is not only one of economics, but also one of conserving an important natural resource. Methods of improving present practices and the search for new treatment and protection are being sought through research. In order to advance such a program, considerable data of a basic nature must be collected such as marine borer physiology, habits, distribution, commensals and parasites, etc.

Advance in the fight against marine borers must be based on scientific research rather than trial and error tests. As one of the largest single users of timber in waterfront structures, the U. S. Navy through its Bureau of Yards and Docks is sponsoring a relatively broad research program. It includes studies in borer habits and distribution throughout the world, carried on by Dr. William F. Clapp, of Duxbury, Massachusetts; physiological studies under direction of Dr. F. G. Walton Smith, Director of the Marine Laboratory, University of Miami, Coral Gables, Florida; and study and evaluation of present and proposed methods of protection at the Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California. Some of the independent investigations are being carried on at the Bernice P. Bishop Museum on observations and surveys of marine borers in the Central and Western Pacific; at the California Academy of Science on ecological conditions affecting distribution of wood-boring mollusks along the Pacific Coast of North America, and on the effects of salinity on Bankia Setecea; at Southern California Marine Borer Council and the University of Southern California working jointly on fouling organisms, Chelura,

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cellulose utilization by _Limnoria,_ predators and commensals of _Limnoria;_
by Taylor-Colquitt Company on a solvent recovery process for use in
wood treating; at University of California, Davis, on ingestion and di-
gestion of wood by marine borers; at University of Washington on occur-
ence of cellulose in _Limnoria Lignorum._

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REFERENCES

Chellis, Robert D., (1948) "Finding and Fighting Marine Borers",
    Engineering News-Record, Part I, pp. 64-67, Mar. 4; Part II, pp.
    94-96, Mar. 18; Part III, pp. 73-76, Apr. 1; Part IV, pp. 73-76,
    Apr. 15.

Hill, C. L. and C. A. Kofoid (1927) (Editors in Chief) "Marine Borers
    and Their Relation to Marine Construction on the Pacific Coast",
    Final Report of the San Francisco Bay Marine Piling Committee.

Jelley, Joseph F., (1951), "More Basic Research Vital in Fight Against
    Costly Marine Borers", Civil Engineering, Vol. 21, No. 4, pp. 200-
    201, Apr.

Long, J. E., (1951), "Subversives of the Sea", U.S. Naval Institute
    Proceedings, Vol 77, No. 2, pp. 171-173, Feb.

Marine Borer Conference (1951), U.S. Naval Civil Engineering Research
    and Elevation Laboratory, Port Hueneme, California, May 10-12.

Meeting to Discuss Marine Borer Situation, (1949), San Francisco, Cal-
    ifornia, jointly sponsored by the California Academy of Sciences
    and the National Research Council, July 20.

U.S. Navy (1949), "The Wayward Worms", Civil Engineering Corps Bulletin,
    Vol. 3, No. 35, pp. 279-282, Oct.

U.S. Navy (1950a), "Harbor Reports on Marine Borer Activities", Bureau
    of Yards and Docks, Research Division, P-43, June.

U.S. Navy (1950b), "Marine Borer Technical Report", Technical Digest, Bureau of Yards and Docks, No. 13, Special Issue.

U.S. Navy (1951a), "Mobile One Seabees Raze Guantanamo Pier", Civil Engineers Corps Bulletin, Vol. 5, No. 50, pp. 3-9, Jan.

U.S. Navy (1951b), "Report on Marine Borers and Fouling Organisms in 56 Important Harbors and Tabular Summaries of Marine Borer Data from 160 Widespread Locations", Navy Docks, TP - Re - 1, April.

PART 2
COASTAL SEDIMENT PROBLEMS

Chapter 9

RECENT GEOLOGY OF COASTAL LOUISIANA

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Recent, in a technical geological sense, refers to the latest
episode of geological time. Definitions vary. Northern European
geologists are likely to refer to about the last 10,000 years for
the reason that only the sedimentary deposits of such an interval
are available for study. Geologists in the United States commonly
regard the Recent as a post-glacial period of somewhat longer
duration. Studies of materials containing the carbon isotope,
C^{14}, are resulting in time determinations in years. Louisiana
geologists define Recent as that period of time during which sea
level made its last general rise. This may have lasted about 30,000
years. Sedimentary layers deposited during that period are regarded
as Recent in age.

The contact between Recent and pre-Recent sedimentary rocks
is a distinct and sharp break along the Louisiana Gulf Coast. Off-
shore, on the continental shelf, and beneath coastal marshes, the
Recent is underlain by a reddish oxidized layer of variable thick-
ness. Cores from wells pass from Recent layers of dark and reduced
silts, clays, and peats, with minor amounts of coarser sediments,
directly into the oxidized zone at the top of the pre-Recent materials
upon which they rest. The organic content of the Recent is compar-
atively high, whereas that of the pre-Recent is much lower. Basal
gravels occur inland at various places but are ordinarily absent
offshore. The most reliable criterion for determining the upper-
most pre-Recent deposits is the oxidized zone at their top.
Oxidation developed as a result of exposure to the atmosphere during
a time of greatly lowered sea level.

The Recent and pre-Recent are separated farther inland by a
topographic break. Recent alluvium fills valleys cut in pre-Recent
materials. Toward the coast the pre-Recent deposits incline gently
under the Recent. The "Prairie" lands of southern Louisiana are
pre-Recent in age. Their surface inclines beneath Recent marsh
deposits in the coastal area. The contact dips at rates as low
as 3 inches per mile. It is considerably steeper in some areas
and shows sharp offsets where displaced by faults at many places.

The pre-Recent topography of Louisiana culminated in an en-
trenched valley system which has been mapped in detail by Fisk
for the U. S. Engineers (1944, 1947, and unpublished reports located
in the New Orleans District Offices and Mississippi River Commission
Headquarters, Vicksburg). The pre-Recent Mississippi flowed along a

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valley over 300 ft. deep where the existing inner boundary of the
coastal marshes is now located. Enormous masses of continental ice
that covered land areas during the last main glacial stage accounted
for subtraction of oceanic waters to such an extent that the univ-
ersal sea level was lowered to the degree indicated by the entrenched
valley system. Low sea level drove shorelines not only downward but
southward, as much as 100 miles in southwestern Louisiana. Projec-
tion of the entrenched valleys along their probable gradients to the
Gulf indicates a lowering of the universal sea level of about 420 ft.

The Recent sediments bottom at a depth of about 550 ft. in
several of the wells of the Louisiana shelf. If 420 ft. of the
section represents a deposit that accumulated as a result of the
last general rise of the oceans, the other 130 ft. represents an
additional deposit required by subsidence of the region during the
Recent. Indeed this coastal subsidence has been occurring, not
only during Recent time but also for such a long interval that a
sedimentary section some 30,000 ft. thick underlies coastal Louisiana.
Rock layers normally incline Gulfward, steeper dips being character-
istic of older layers because they have shared the effects of coastal
subsidence for greater lengths of time. Rocks that were formed on
flood plains, coastal marshes, deltas, and shallow sea floors now
lie below the Louisiana Coast at depths ranging down to almost three
times that of the deepest water in the Gulf of Mexico.

RECENT SEDIMENTS

Two main sources account for the materials found in the Recent
sediments: (1) adjacent land, and (2) biologic. The Mississippi
River and other streams bring most of the mineral and rock fragments
that lodge as sedimentary deposits in the coastal region, together
with colloidal materials and dissolved salts. The biologic contri-
bution is overwhelmingly that of carbonaceous plant remains in the
marshes and primarily that of calcareous animal remains offshore.

The larger rivers brought enough sedimentary matter to fill
not only their own pre-Recent trenches but also to extend their
flood plains Gulfward as deltas. Estuarine mouths occur in cases
where the rise of sea level was too rapid to be balanced by the
sedimentary loads at hand. Mobile Bay is a conspicuous example, as
are several indentations along the coast of Texas. The Mississippi
lies at the other extreme. During the latter part of Recent time
it has had enough surplus sediment to build several deltas. Remnants
of its Teche and still older deltas lie somewhat inland and westward
of the present-day valley mouth. Later the Terrebonne-Lafourche
delta system accounted for an expansion of land area having about

the shape and size of the delta of the Nile. More recently a large
delta was built eastward from the site of New Orleans. Finally came
the delta which is growing so actively today, the Balize Delta,
southeastward from New Orleans. The delta-building period may
represent over ten thousand years and during at least its first
half the Mississippi generally discharged in a westerly direction.

LAND FORMS

The natural levee is the fundamental depositional form of
coastal flood plains and deltas of Louisiana. It is a belt of some-
what elevated land along the side of a channel. Typically it is
firm land consisting of silt and other comparatively coarse materials
carried by streams and deposited at times when flow occurs across
banks. In the coastal marshes pairs of natural levees mark positions
of both actively flowing, silt-depositing streams and abandoned
streams that once had such characteristics. The system of natural
levees associated with each of the modern Mississippi River deltas
resembles somewhat the radiating fingers of a spread hand, or the
ribs of some leaves. Between the natural levees are basins which
typically widen Gulfward. Along the coast these contain shallow
bays, such as East Bay and West Bay to the sides of the natural
levees of Southwest Pass of the Mississippi. Farther inland they
are marsh basins which may still contain conspicuous bodies of
water, such as Barataria Bay, Little Lake, Lake Salvador, and Lac
des Allemands, all of which lie between the natural levees of the
Mississippi and those of Bayou Lafourche and other older courses of
the Mississippi. The Atchafalaya and other basins are tree-covered
swamps still farther inland. Natural levees rise to levels set by
floods under natural conditions. In central Louisiana they stand
15 ft. or more above basin floors and toward the coast as little
as a few inches. The general level of land in the coastal marsh
basins is that of mean high tide.

Most coastal marsh area is basin. Older basins in western
Louisiana are so filled that their surfaces are rather firm over
wide areas. Younger basins in and between the various deltas to
the east contain greater areas of bay, lake, pond, or soft-surfaced
marsh. Silty natural levee deposits are found at various depths in
the basins by probing or boring through overlying deposits which
are ordinarily highly organic. Layers of silt and fine sand
alternate with peats and clays under the marshes both because older
and now submerged patterns of delta channels are encountered and
because various old lake floors or bay bottoms are present. In detail,
the subsurface section is rather complicated. The intricate patterns
of today's surface in the vicinity of the Mississippi mouths are

similar to those revealed by detailed subsurface studies in the older areas to the west. Flocculated colloids form deposits of firm blue clay toward the coast. Outer marshes are notably firmer than those inland.

Pitted against the forces of land enlargement are the erosional forces of the Gulf. Wave attack drives shorelines inland, opposing the effects of deposition or accumulation of plant remains. Waves and currents accumulate the coarsest materials available as beaches along the shore. Finer materials are shifted outward, toward deeper waters. Where sand is available beaches are sandy. Such is the case along limited parts of the Louisiana coast. In other cases the coarsest materials are shells, bottles, pieces of iron, and flotsam in general. Predominantly, however, Louisiana beaches are composed of silt and very fine sand. In exposed situations to the east of the Mississippi River many consist almost entirely of coarse shell. Westward, shells are smaller, less conspicuous, and more fragile, excepting on shell reefs where oysters predominate. Most of the reefs are located near the mouths of streams, either active or abandoned.

The main result of delta growth as it affects shoreline patterns is the development of a highly irregular coast. That of erosion is to smooth irregularities and develop long, straight beaches.

FOUR TYPES OF COAST

Louisiana exhibits four distinctly different types of coast. Each is the resultant of the two main controls of shoreline patterns, deposition and erosion.

Plaquemines shoreline is the irregular type existing toward the mouth of the Mississippi River. Natural levees protrude out across shallow bottoms to form the radiating passes through which river water reaches the Gulf. Where levees rise a few inches above the level of spring tides they are densely covered by tall grass, willows, and shrubs. Where high enough to escape flooding during intervals of several years large hackberries, oaks, and other trees become firmly established. The natural levees continue beyond the shoreline as submerged features with crests marked by stranded logs and commonly by the presence of myriads of pelicans and other birds standing in shallow water. Main Pass and the passes directed eastward have pronounced underwater natural levees. Jetty construction and deeper waters ahead of South and Southwest Passes discourage the concentration of sedimentary deposits as natural levees.

Wave action and near-shore currents concentrate coarse silt and fine sand both as bars near the outer ends of jetties and as long more or less submerged spits and bars that surround most of the Balize Delta. Exposed parts of spits become beaches that are most conspicuously developed at right-bank pass mouths. The shallowness of the circumferential bar precludes navigation by deep-draft boats into any but the two jettied passes. The bays between passes are sheltered by these outer shallow bottom fringes.

The general plan of bird-foot delta is modified by crevasses which have poured water and sediment into the bays or basins between natural levees. Webs of alluvial deposits are thus formed between the talons of natural levees. The bird-foot pattern grows more and more that of a duck foot. Garden Island Bay, between South and Southeast Passes, has been almost completely filled since 1890. Extensive areas of marsh have been added below The Jump and in the entire territory to the east and north of Head of Passes within recent years.

St. Bernard shoreline is the highly complicated type to the east of New Orleans. Bayou La Loutre, the next older main Mississippi River channel to the one in use today, was abandoned more than a thousand years ago. The extensive delta it built eastward was deprived of supplies of inorganic sediment but continued to be affected by Gulfward tilting, so that more than one half of its original area is now submerged. Patterns of distributary streams and pass ramifications remain in the marshes. Toward their distal ends natural levees have become double-islands; strips of land flanking filled and abandoned channels. Eastward the levees pass below the waters of Chandeleur and Breton sounds. The basins between natural levees are now the large shallow bays and harbors of the region. Wave erosion is modifying their outlines and resulting in a comparatively rapid spread of water surface at the expense of former marsh. Twenty miles and more east of the ragged fringes of the mainland is the beach. Coarse materials originally concentrated around the St. Bernard Delta coast have been driven shoreward to form a continuous arc, the Chandeleur Islands, with southwestward extensions leading to Breton Island and associated bars. The original extent of the St. Bernard Delta is unknown, but Indians lived well beyond the outer face of the Chandeleurs because fragments of their pottery and an abundance of comparatively freshwater shells (Rangias) are lodged by waves among the coarse shell and sandy materials of the beach. Large blocks of peat are detached by waves and carried to the beach.

Centrally within the Chandeleur arc are a few small islands representing remnants of the original marsh of the St. Bernard Delta.

Though the actual silts of natural levees lie about 20 ft. below
their surfaces, island shapes reflect channel patterns that origin-
ated on marsh surface. This feature is found above many submerged
marshes. The more luxuriant vegetation of the natural levee crest
tends to perpetuate itself even though its original cause has
subsided well below plant roots. The roots themselves form a
somewhat more firm base for establishment and maintenance of
succeeding vegetation than exists in or above original marsh basins.

Mangrove swamps flank much of the inner part of the Chandeleur
arc. The outer beach migrates over and across the mangroves so
rapidly that after a short period of burial which results in
killing all plants their stubs and roots remain exhumed on the
outer beach in density equal to that of the original swamp. Mangrove-
held beach is decidedly more resistant to wave attack than beach
without such reinforcement. Indentations along the outer beach
mark places where bayous ran through the original mangrove swamp.

Terrebonne shoreline lies to the west of the Balize Delta and
reflects the presence of an abandoned delta system closely resembling
that of the Nile. The forces operative in producing the complications
of the St. Bernard shoreline have acted so long that the outer beach
has migrated landward to become tangentially attached to the mainland
fringes. Sandy beach predominates. Between Grand Isle and Timbalier
Island the eastern half of the sand beach is now moving into almost
continuous marsh. The western part is invading bays and waters
between the Lafourche and Terrebonne subdeltas. During the last
fifteen years amazing changes have occurred in this area. A few
scattered islands of sand have become united into a broad strip
about ten miles long which terminates inland and to the west of
Timbalier Light. In few places are quadrangle maps issued so
recently as 1935 so obsolete. Pottery scattered along this entire
coast attests to the fact that Indians were living on lands located
several miles out in what is now the Gulf of Mexico. Truncated
lakes and stream patterns reflect the rapidity of shoreline advance
into marshes. Comparisons of surveys indicate such rates of beach
advance as averages of well over 100 feet per year. Great changes
mark times of severe storm. The ordinary year commonly witnesses
widening of beaches Gulfward even though the general direction of
beach movement is landward at a spectacular rate.

Subsidence of the Terrebonne area has resulted in extensive
development of flotant; floating marshes overlying either water or
soft ooze which are more or less passable to a man on foot. These
form in basins extending well inland into areas of fresh-water
marsh. During hurricane waves, when broad areas of marsh may be
covered to depths such as 6 ft., floating marsh normally rises

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intact. Now and then some part breaks away through buoyancy and drifts away. It was thus that Wonder Lake, some 15 mi. southeast of Houma, formed in 1915. This peculiar, somewhat star-shaped, break in the flotant, has axes more than 1.5 miles long.

Cameron shoreline extends westward from Marsh Island into Texas. Processes of erosion have long held the upper hand. The shore is smooth and beach is practically continuous. Coastal sediments are derived principally from marsh which is under erosional attack rather than from supplies furnished by active streams. Waves nearly everywhere are eroding clay or peat. Such silts, sands, or other coarse materials as are available are incorporated into the beach, but they are insufficient in amount to produce the firmness characteristic of outer beaches to the east. Where currents bring large amounts of colloidal and other very fine materials large ooze flats form in front of the beach. A powerful boat with a draft of as much as 6 ft. may churn through such ooze to within a few feet of the land at some places. The development of ooze flats is occurring at a spectacular rate to the west of the artificial Wax Lake outlet of the Atchafalaya Basin.

There is a long and complicated history in the Cameron marshes. Cheniers are inland beaches dating from times when the Gulf beaches advanced into the marshes only to be followed by times when marshes advanced toward the Gulf. Beaches were left stranded behind marshy flats. The alternations between conditions of coastal advance and retreat were caused by alternations between surplus and deficient supplies of sediment along the coast. During Teche-Mississippi times, several thousand years ago, the Cameron coast at times was more than amply supplied by sediments from westward passes of the river. Coasts grew out Gulfward in the manner now characteristic of the Balize Delta. But at other times the river experienced diversions toward the east, so that what is now the central Louisiana coast received most of the sediment and the western coast experienced dominance of wave attack, beach formation, and landward migration. Beaches formed at such times are now cheniers. Later periods of surplus sediment and marsh development account for the deposits south of each chenier. More recent cheniers truncate those of earlier date at several places, just as the present-day shoreline truncates Chenier au Tigre. The last main ridge, Grand Chenier and extensions eastward through Pecan Island and westward to Cameron, reaches elevations nearly 10 ft. above Gulf level and is composed of fresh sand and shell. At a maximum it approaches a distance of nearly 10 mi. from the present-day beach. Older cheniers lie inland and are more altered and submerged in accordance with age. The modern beach is being converted into a chenier in places where Atchafalaya sediments are building ooze flats along the coast. These flats will evolve into marshes extending Gulfward from the chenier.

CHANNEL PATTERNS

The meandering cheracteristic of the Mississippi River disappears toward the coast and most particularly where the channel is cut in clay. Though many theories have been advanced to account for the "bird-foot" pattern of the Balize Delta, there is little merit in any of them, with the exception of the idea that the river is confined to a single channel for a long distance below New Orleans for the reason that it runs through clay. This clay was deposited offshore during the times when older deltas were forming. The Teche and subsequent deltas, including the St. Bernard, extended into the Gulf across areas without notable clay deposits and for that reason branched freely to form distributary patterns more like those of the Nile Delta.

The patterns of channels below crevasses are anastomotic. Garden Island Bay and other similar areas are characterized by comparatively straight, but freely branching channels that surround lenticular islands. Surplus sedimentary load exists for the low velocities encountered as currents reach bays.

As a general rule distributary channels of old main river courses are comparatively straight. This pattern is retained after abandonment. As a group they are the channels that deteriorate most rapidly after abandonment. Distributaries shoal rapidly immediately after diversions lead waters elsewhere and they have drainage basins not appreciably wider than their own widths, because the broad slope from each of their natural levee crests is outward into flanking basins. In contemplating a route for a journey by skiff from aerial photographs or detailed maps of the marshes one generally seeks avoidance of straight channels unless they retain inflow from some active source.

Each marsh basin develops its own drainage net. The streams are sinuous and in many cases meander so rapidly that they exhibit numerous cut-offs. These are the waxing and deepening streams of the marshes. The deepest and most developed lead from lakes that fill or empty according to tidal levels in the Gulf. The tidal channels are without appreciable natural levees, though a slight firmness along their banks contrasts with conditions farther out in the marsh basins. Limited segments of straight channels commonly become incorporated in the tidal-channel network and are deepened or widened accordingly.

CANAL LOCATIONS

The history of relocations of the Intracoastal Waterway across Louisiana is illustrative of a need among engineers for geologic

advice. Older routes utilized lakes as far as possible and lay well to the south of the present waterway at most places. Maintenance costs were found to be excessive in lakes, so that canals are now dug around them, even though distance is appreciably increased, as in the case of Lake Salvador. Maintenance costs are also excessive if canals are bottomed in peat or other soft materials. The canals most readily maintained lie well toward the northern boundary of the Recent coastal deposits, where the underlying pre-Recent forms the bottom and fair proportion of excavated bank. The Intracoastal Waterway now approximates the inner boundary of the Recent for long distances.

Canals leading to the Gulf are almost impossible to maintain if they reach a portion of the coast where ooze accumulates in quantity and where the coast is building outward. A proposed canal which would utilize the existing but unuseful Freshwater Bayou Canal at the southernmost projection of the Louisiana coast to the west of Marsh Island represents about the worst possible condition. Long jetties and endless dredging would be required to maintain navigability. The most desirable places for canal mouths to terminate are those where natural agencies maintain deep water, as in the deep passes leading into Barataria and Vermilion bays. Where other sites must be selected the most desirable are places where coastal beaches are moving inland at comparatively rapid rates. Detailed plans should take into account a prevalence of westward drift of debris.

RECLAMATION PROJECTS

Somewhat more than fifty agricultural reclamation projects have been undertaken in the marshes. The general pattern is that of constructing low levees around fields that are ditched to sumps from which water is pumped. Practically all have failed for the reason that plowing and cropping lowers the surface rapidly. Pumping costs increase and drainage difficulties grow more acute until financial failure occurs. Old fields located near highways have been converted into fishing ponds with some success.

One successful undertaking has been the construction of levees protecting higher land against salt water encroachment. The orange groves along the lower Mississippi and various fields in the western part of the state have benefited from such protection, especially at times of high water during storms. The groves lie between artificial levees on Mississippi natural levee crests and a second set of levees toward the marshes.

The principal uses of the marshes are for muskrat and nutria trapping and for cattle grazing. The yield from the best trapping

lands exceeds that of nearby agricultural land. Canals and other engineering structures commonly upset vegetational patterns and thus interfere with pastures. Water bottoms toward the coast with salinities favorable to oyster production are also valuable and are subject to deterioration or improvement when canals and ditches are run through adjacent marshes.

The necessity for extreme caution exists whenever construction projects are likely to affect the water table either within the marshes or in adjacent pre-Recent territory to the north. Industrial and agricultural demands have already lowered water tables to danger points in several places, so that significant salt-water encroachments have occurred. The only considerable area where shallow artesian conditions remain within Louisiana marshes is eastward, in the St. Bernard Delta. The water is somewhat brackish.

It is certain that an increased number of salt-water locks will be necessary for purposes of maintaining adequate trapping and grazing land as well as for protection of more inland places against encroachment of brackish and saline ground water.

REFERENCES

Fisk, H. N. (1944) Geological Investigations of the Alluvial Valley of the Lower Mississippi River: Miss. Riv. Comm., Vicksburg, 78 pages, 33 pls.

_____(1947) Fine-grained alluvial deposits and their effects on Mississippi River activity: Waterways Exp. Sta., Vicksburg, 82 pages, atlas of 74 pls.

Russell, R. J. (1936) Physiography of Lower Mississippi River Delta, La. Geol. Surv., Bull. 8, pp. 1-199, 19 figs.

_____(1939) Louisiana stream patterns: Bull. Am. Assoc. Petrol. Geols., vol. 23, pp. 1199-1227, 7 figs.

_____(1940) Quaternary history of Louisiana: Bull. Geol. Soc. Am., vol. 51, pp. 1199-1234, 4 figs.

_____(1942) Flotant: Geographical Review, vol. 32, pp. 74-98, 12 figs.

_____(1948) Coast of Louisiana: Bull. de la soc. belge. de Geol., de Paleontol., et d'Hydrol., vol. 57, pp. 380-394, 5 figs.

_____and Howe, H.V. (1935) Cheniers of Southwestern Louisiana, Geographical Review, vol. 25, pp. 74-98, 5 figs.

Chapter 10

SEDIMENTATION AT THE MOUTH OF THE MISSISSIPPI RIVER

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Sedimentation at the mouth of the Mississippi River is a phenomenon that has been under study by the Corps of Engineers, Department of the Army, during the past 120 years. The primary objective in these investigations has been the determination of the most economical method of maintaining required navigation depths through the Mississippi River Passes for ocean-going vessels that serve the Ports of New Orleans, Baton Rouge, and indirectly the vast Mississippi Valley river traffic.

The objective of this paper is to summarize pertinent sediment investigations of the Mississippi River and Outlets below the latitude of Old River, Louisiana, and to discuss the complex nature of the sedimentation problem in the Mississippi River delta area. Topographic features of the region are shown on Figure 1.

INTRODUCTION

GENERAL

The Mississippi River rises in Lake Itasca, Minnesota, and flows in a general southerly direction to the Gulf of Mexico, a distance of approximately 2,434 miles. The river, with its tributaries, drains about 41 percent of the area of the United States proper. This drainage basin of 1,245,000 square miles, including all, or parts of 31 states and two Canadian provinces is shown on Figure 1. The Mississippi is navigable to ocean-going commerce as far upstream as Baton Rouge, Louisiana, 247 miles from the Gulf of Mexico via Southwest Pass.

New Orleans, one of the major ports of the country, with a population of about 600,000, is located on the river about 114 miles from the Gulf via Southwest Pass and 107 miles by South Pass. The principal industries of New Orleans are closely allied with river traffic. Manufacturing plants covering a wide range of products are located at that site in order to utilize the raw and semifinished products which are handled through the port or produced in the adjacent territory. This city is also an important terminal and point of origin for barge shipments to the east and west via the Intracoastal Waterway and north or south on the Mississippi-Ohio River system.

DESCRIPTION OF THE LOWER MISSISSIPPI RIVER

The levees of the Mississippi River below the latitude of Old River, Louisiana, are built relatively close to the banks of the river and extend downstream on the west bank to Venice, La., 10 miles above the Head of Passes, and on the east bank to Pointe-a-la-Hache, 44 miles above the Head of Passes, and thence from Mile 33 to Mile 11.5 above Head of Passes. No important tributaries enter the Mississippi River between Old River and the Head of Passes, all the drainage being carried to the Gulf by adjacent
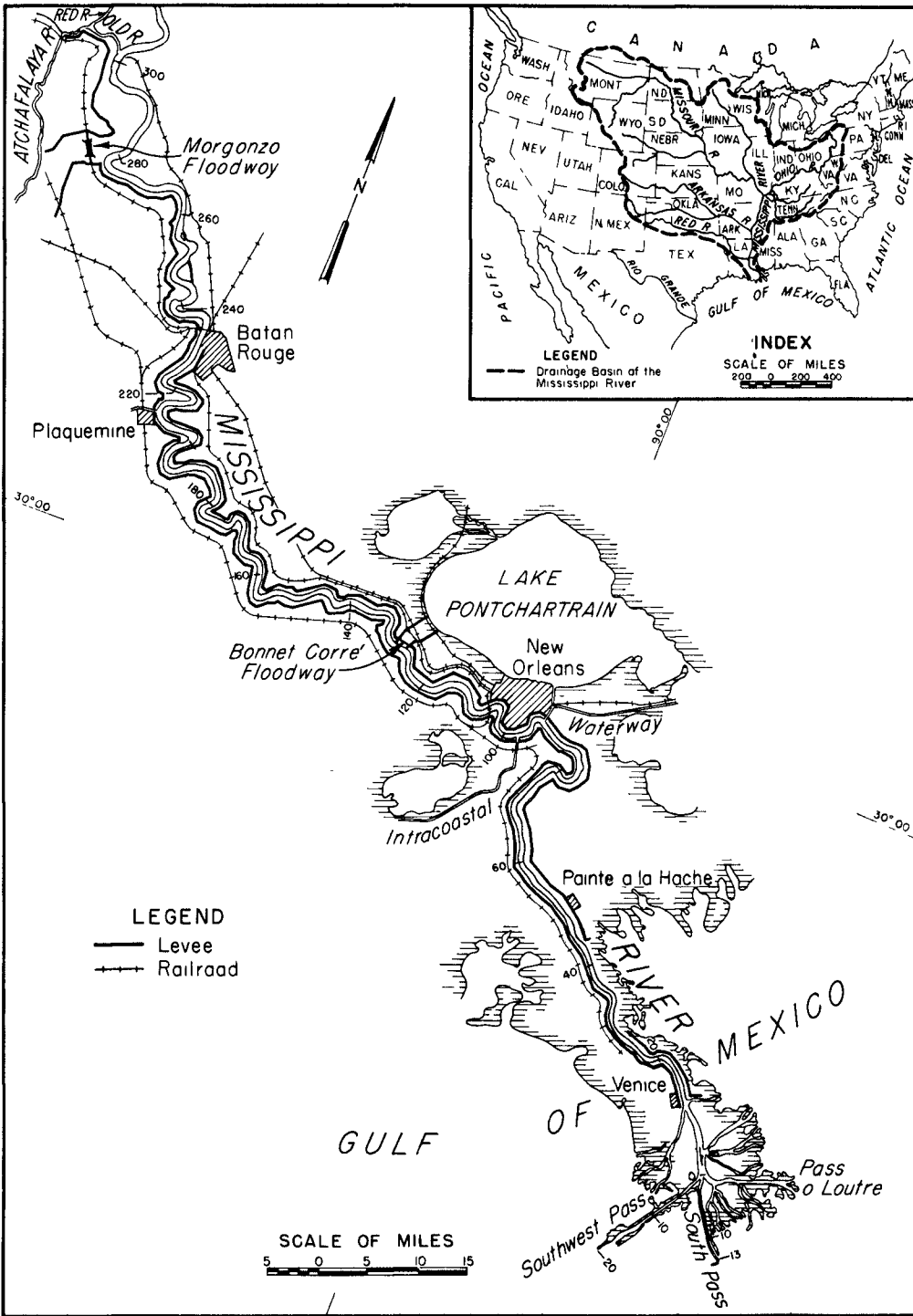
Fig. 1
General map, Mississippi River from Old River, La. to the Gulf
of Mexico; and, drainage basin of the Mississippi River.

streams or lakes.

Approximately 322 miles from the Gulf of Mexico, the Mississippi River is connected on its west bank through Old River to Red River, a tributary, and to the Atchafalaya River, a distributary, connecting with the Gulf approximately 125 miles west of the Mississippi River Passes. The direction of flow in Old River may be in either direction, depending on stage differential between the Red and Mississippi Rivers, however, its flow at the present time is generally westward from the Mississippi to the Atchafalaya practically all of the time.

Above Baton Rouge a 9-foot channel is maintained. In this reach is located the Morganza Floodway, a part of the Mississippi River Flood Control Project. This floodway, having a design capacity of 600,000 c.f.s., is located on the west bank of the Mississippi River, approximately 25 miles northwest of Baton Rouge, Louisiana. It is an overbank floodway confined by guide levees with a control structure at the upper end along the Mississippi. The floodway is designed to pass flood waters from the river into the Atchafalaya Basin in order to limit flow in the Mississippi River below Morganza, La., to the safe capacity of the leveed river channel, 1,500,000 c.f.s. This floodway, now under construction, has never been operated.

Between Baton Rouge and the Port of New Orleans, a distance of approximately 132 miles, except for two or three local areas where shoaling occasionally occurs during times of flood, the channel is never less than 40 feet in depth; the minimum channel width normally is about 500 feet, while the average width exceeds 1,000 feet. The second controllable floodway, the Bonnet Carre Spillway, is situated on the east bank of the Mississippi, about 30 river miles above the City of New Orleans. It consists of guide levees with a control structure at the upper end along the Mississippi. This floodway is designed to divert 250,000 c.f.s. of river water to the Gulf of Mexico via Lake Pontchartrain in order to limit flow in the Mississippi past New Orleans to about 1,250,000 c.f.s. and was operated for that purpose in 1937, 1945 and 1950.

Below New Orleans, the river flows in a generally southeasterly direction a distance of approximately 95 miles to the Head of Passes. The gap in the east bank levee system between miles 44 and 33 above the Head of Passes is known as the Pointe-a-la-Hache Relief Outlet or the Bohemia Spillway. This uncontrolled State of Louisiana project is rapidly deteriorating as a highwater relief outlet as a result of sedimentation and lack of required maintenance. The purpose of this project was to lower Mississippi River flood stages at and below New Orleans. At low water, the river cross section varies between one-half a mile and a mile in surface width and between 50 and 200 feet in depth.

For approximately eight months of the year the lower Mississippi River is influenced to a greater or lesser extent by flood waters. Under average conditions, the river starts to rise in December, reaches a crest in April and recedes to low water stages by the end of August.

The delta at the mouth of the Mississippi River is typical of all alluvial silt-bearing streams. At the present time, the three principal
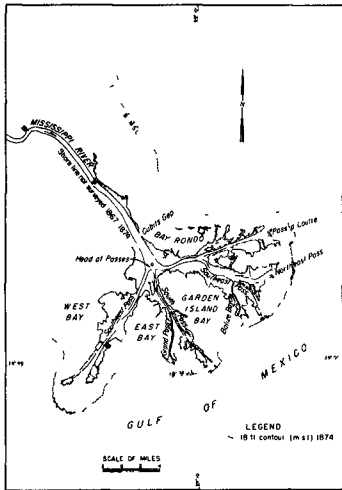
Fig. 2. Mississippi River Passes, 1874 conditions as developed by the U.S. Coast and Geodetic Survey.
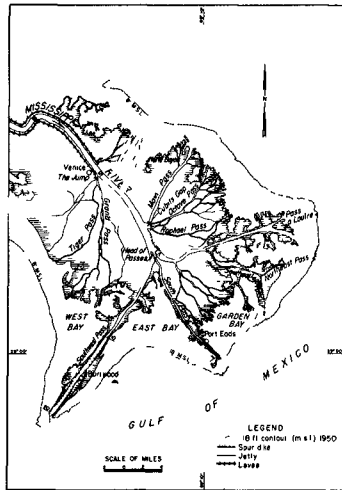
Fig. 3. Mississippi River Passes, 1950 conditions as developed by the U.S. Coast and Geodetic Survey, and as revised by the Corps of Engineers.
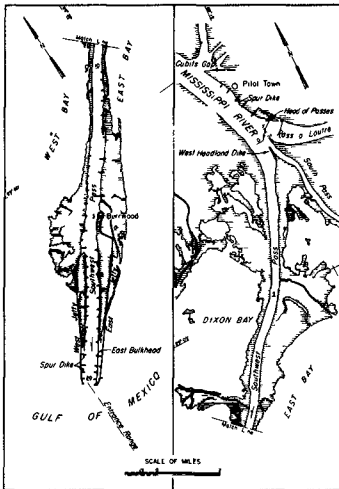
Fig. 4. Improvements of the Southwest Pass, Mississippi River.
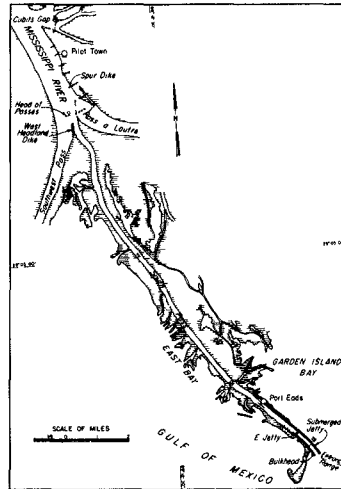
Fig. 5. Improvements of the South Pass, Mississippi River.

outlets of the river below New Orleans and in the order of their discharge capacity are Pass a Loutre, Southwest Pass and South Pass, which are joined at the Head of Passes. All of these passes have been used for commerce at various times, but only South and Southwest Passes have been artificially improved to permit their use by ocean-going ships. The remainder of the flow is discharged from the river upstream from the Head of Passes through Baptiste Collette Bayou, The Jump and Cubits Gap.

Pass a Loutre, the largest of the principal passes has a length of about 15 miles, flows slightly north of east, and discharges through several outlets to the Gulf. Southwest Pass, the most westerly of the three main outlets, has a length of 21.2 miles and flows southwesterly to the Gulf. South Pass, the smallest of these passes, has a length of 14.2 miles and flows in a southeasterly direction to the Gulf. The maximum discharges in c.f.s. recorded for these passes during the 1950 flood cycle were 410,000, 354,000, and 166,000, respectively.

Other outlets of lesser importance include Baptiste Collette Bayou, a small opening in the east bank about 83 miles below New Orleans; The Jump, an outlet through the west bank about 84 miles below New Orleans; and Cubits Gap, in the east bank of the Mississippi about 91 miles below New Orleans. During the 1950 high water the maximum discharges in c.f.s. through these openings were 28,700, 35,100, and 151,800, respectively.

GEOGRAPHY OF THE PASSES.

The land adjacent to the Mississippi River in the vicinity of the passes has undergone extensive changes during its period of record. The natural tendency of the Mississippi as a silt-bearing and delta-forming stream is to throw off branches from the main stem, which, in turn, subdivide into smaller branches. The process of subdivision continues until the Gulf is reached, with the individual outlets of small capacity when compared to the main river.

The banks of these passes are subject to overflow from time to time during flood periods. Materials in suspension are deposited in quantities sufficient, generally, to maintain height of banks at a rate equal to the subsidence of the delta, a rate of about 0.1 foot per year. A comparison of Figures 2 and 3 shows that considerable new ielta has been formed in the area of the passes during the period 1874 to 1950. Bay Ronde has completely disappeared and in its place are deltaic deposits traversed by numerous minor outlets. On the other hand, excessive subsidence and loss of land area have taken place in East Bay, as a result of closure of many outlets along Southwest and South Passes that formerly discharged into the bay. Small regulated outlets are provided at critical places to maintain the banklines by supplying the adjacent bay areas with silt-laden water during times of flood.

IMPROVEMENTS OF THE PASSES

In view of the importance of a deep entrance channel to the development of the City of New Orleans, early efforts were made to deepen the natural 9-foot depth over the bars at the mouths of the Mississippi River, commencing as early as 1726 by the French government, and continuing

intermittently for over 100 years. None of these efforts met with much success.

After considerable controversy as to the best method to be followed in securing a satisfactory entrance channel from the Gulf to the Mississippi River, Congress, in 1875, accepted the "no cure, no pay" proposition of Captain James B. Eads wherein he was to secure, by construction of jetties at the mouth of South Pass, a 30-foot channel through the pass and maintain it for a period of 20 years. The results obtained far exceeded expectations. The jetties were completed in 1879 and a 30-foot channel was secured and maintained by dredging with brief interruptions during seasons of high floods, until 1901, when the work of maintenance was taken over by the Corps of Engineers.

As a result of the successful improvement of South Pass, the commerce of the Port of New Orleans rapidly increased and another deeper channel was required to meet the growing demands of navigation. Jetty construction in Southwest Pass was started in 1904 and completed in 1908. The east and west jetties extended to 10 or 12 feet of water and were 21,000 and 15,000 feet in length. Depths were increased from 9 to 20 feet. Over the years since the initial construction, additional improvements of various types have been undertaken to maintain the required project depth. These works included jetty extensions of some 3,000 feet, deflection of the axis of the bar channel some 35 degrees east of the axis of the jetty channel; contraction works to narrow the channel and a considerable amount of dredging.

EXISTING NAVIGATION PROJECT ON THE LOWER MISSISSIPPI

The navigation project on the lower Mississippi, as authorized by the River and Harbor Act of 2 March 1945 and prior Acts, provides for channel dimensions below mean low Gulf level of: Baton Rouge to New Orleans, 35 feet deep by 500 feet wide, 128.6 miles long; within limits of the Port of New Orleans, 35 feet deep by 1500 feet wide, 17.2 miles long; lower limits of the Port of New Orleans to the Head of Passes, 40 feet deep by 1000 feet wide, 86.7 miles long; Southwest Pass, 40 feet deep by 800 feet wide, 21.2 miles long; Southwest Pass Bar Channel, 40 feet deep by 600 feet wide; South Pass, 30 feet deep by 450 feet wide, 14.2 miles long; and South Pass Bar Channel, 30 feet deep by 600 feet wide.

The project is complete except for construction of contracting dikes, screening of existing dikes, and deepening of the pass and bar channel from 35 to 40 feet in Southwest Pass. Figures 4 and 5 show relative locations of improvements.

HISTORY AND DESCRIPTION OF SEDIMENT INVESTIGATIONS

The history of sediment observations on the lower Mississippi River dates from 1838. These observations, the first of record in the United States, were made in connection with a survey of the mouths of the Mississippi under the direction of Captain A. Talcott. Results indicated 580 parts of sediment per million parts of water in South Pass. In Southwest Pass, 18 samples from the surface and 9 samples from below the surface gave a combined sediment concentration of 796 p.p.m.

SEDIMENTATION AT THE MOUTH OF THE MISSISSIPPI RIVER

During the period from 1843 to 1846, Professor J. L. Riddell obtained samples from the river at New Orleans. The average sediment concentration was found to vary from 804 p.p.m. to 864 p.p.m. Other measurements were made in 1851-1852 by Professor C. G. Forshey at New Orleans.

In connection with improvements to South Pass, the Corps of Engineers sampled in this location continuously from 1877 to 1898. At the middle of the pass, samples were taken at the surface, 8, 16 and 24 feet below the surface, and near the bottom. About 150 feet from each shore, samples were taken at the surface, mid-depth, and near the bottom. The results of sediment observations in South Pass for each of the years from 1879 to 1893 are published in the annual report of the Chief of Engineers for 1894. The maximum sediment concentration was 1,100 p.p.m. in 1888, the minimum was 456 p.p.m. in 1879, and the mean was 688 p.p.m. The maximum concentration of sand was 473 p.p.m. in 1880, the minimum was 115 p.p.m. in 1879, and the mean was 313 p.p.m.

During the period 1879 to 1880, extensive sediment investigations were conducted under the direction of the Mississippi River Commission. Sediment observations were conducted at the Carrollton range, New Orleans, Louisiana, for a period of 264 days from December 19, 1879 to October 8, 1880. The work comprised 696 specimens collected in 29 sets of observations. Samples were taken at the surface, at mid-depth, and near the bottom at each of eight points located at about equal intervals across the sediment gaging range. These samples were combined vertically and horizontally in separate bottles. The vertical combination for any station contained two ounces at each depth placed in one bottle, so that eight bottles, each containing six ounces were required for one day's samples. In the horizontal combinations, two ounces from each of the surface samples were placed in one bottle, two ounces from each of the mid-depth samples were placed in a second, so that three bottles each containing 16 ounces were required for the day's samples. The points at which samples were taken were fixed by intersecting range lines.

During the period January 8 to August 5, 1927, two surveys and sets of observations were made by direction of the Mississippi River Commission at the Pointe-a-la-Hache Relief Outlet to determine the effect of the operation of the spillway upon each element of the Mississippi River regimen; i.e., high water slope, scour and fill in the river bed, volume of water discharged, and quantity of sediment carried in suspension. During this period, a total of 1,584 samples were collected in 33 sets of observations spaced at intervals of about four to five days. In each set of observations, samples were taken at the surface, mid-depth and near the bottom at eight points located at about equal intervals across two sediment gaging sections, one of which was located about 4.1 miles above the upper end of the spillway and about 1.0 mile below the lower end.

In 1929, under the direction of the Mississippi River Commission, sediment observations were made throughout the year at specified locations that included the taking of sediment samples at Red River Landing, Louisiana, and at the Carrollton range, New Orleans, Louisiana, with a procedure similar to that employed in 1927.

The Corps of Engineers collected numerous samples from the lower Mississippi in 1930-1931. Samples were generally taken about three times each

week ~t the surface, mid-depth, and near the bottom in each of eight verticals. From the average of thousands of discharge measurements taken previously in the Mississippi River, it was found that the upper quarter, the middle half and the lower quarter of a given cross section carried 27.2, 51.2, and 21.5 percent, respectively, of the total water discharge. Hence, weights of 1, 2, and 1 were applied to values of suspended sediment concentration observed at the surface, mid-depth and bottom layers, respectively.

In 1938 the Corps of Engineers collected samples and obtained velocity measurements simultaneously in the passes near the mouth of the river with six to eight verticals of eight to twelve points each.

Currently, suspended sediment sampling is being conducted on the lower Mississippi River at the Baton Rouge, Louisiana, discharge range. Forty point samples are secured at each observation at about a rate of twice monthly during the low water season and twice weekly during the high water season. The sampling points are spaced transversely and vertically so as to represent sections of equal discharge. Eight verticals are spaced across the width of the stream with five points at depths along each vertical. The location of these points are made according to a relatively simple graphical method devised by E. W. Lane of the Bureau of Reclamation. Based on the water discharge distribution over the desired range in stage, this method can be applied to any gaging station where sediment and water discharge measurements are taken regularly. The results of the observations at Baton Rouge initiated in 1949, provide an excellent basis for determination of suspended sediments carried by the Mississippi.

FACTORS AFFECTING SEDIMENTARY ACTION IN THE PASSES

Factors affecting sedimentary action in the passes may be divided into two classes, natural and artificial. The natural class includes such factors as river discharge, littoral currents, salt water intrusion, tides and wind direction. Artificial factors include the man-made works of diversion structures, contraction structures and dredging. The interrelation of these factors is a complex process with one influence modifying the influence of others in varying degrees of intensity from time to time.

Sedimentary action that causes shoaling involves three basic processes: pickup of material in one area, its transportation to another area, and its deposit in that other area to form a shoal. Thus, all of the above-mentioned factors may at times contribute to the shoaling of an area and at other times serve to move it, either to another part of the area under consideration or entirely out of the area. At or near the mouths of the passes the tidal influences are very important considerations in studies of shoaling action with, of course, the fluvial characteristics of the river tending to predominate over the tidal characteristics upstream from the mouth.

NATURAL FACTORS

Mississippi River flow at New Orleans is the forecast point for hydraulic characteristics in the passes. General criteria concerning the

natural factors affecting sedimentary action in the passes are as follows:

River Discharge - At mean low water, about 3 feet above mean sea level on the Carrollton Avenue gage, the discharge at New Orleans, 103 miles above the Head of Passes, is about 300,000 c.f.s. At normal stages, about 9 feet above MSL on the Carrollton gage, the discharge is about 600,000 c.f.s. At flood stage, about 17 feet above MSL on the Carrollton gage, the discharge is about 1,000,000 c.f.s. The minimum recorded discharge at New Orleans is 79,000 c.f.s. and it is estimated that, with the Morganza and Bonnet Carre floodways above New Orleans in operation, the maximum discharge will be limited to 1,250,000 c.f.s. This total discharge is normally distributed through the passes in the percentages of: Southwest Pass, 30; South Pass, 15; Pass a Loutre, 40; and Cubits Gap and other outlets, 15. The normal range in stage at New Orleans is about 16 feet, from about 2 to 18 feet above MSL on the Carrollton gage. The maximum allowable stage is 20 feet.

Velocities through the three principal passes range up to about 6 feet per second. Mean velocities at the heads of these passes usually range from about 2 to 5 feet per second.

Littoral currents - Relatively strong littoral currents, about 1 foot per second, flow generally in an east to west direction just Gulfward of the passes.

Salt water intrusion - Observation and study of salinity conditions in the lower Mississippi reveal that below certain fresh water discharges from the upper river there is intrusion of dense salt water from the Gulf of Mexico through the deep passes, South and Southwest Passes. The intrusion is in the form of a wedge having a fairly well defined interface which moves upstream through the passes below the outward fresh water flow. At stages of 10 to 12 feet above MSL on the Carrollton gage, or about 800,000 c.f.s., the salt water wedge is held just outside of the outer end of the jetty channel by the strong river current. At mean low water, about 3 feet above MSL on the Carrollton gage, or 300,000 c.f.s., the salt water interface is normally located at the Head of Passes. At extremely low stages, below 3 feet, the salt water proceeds upstream, the extent depending on the amount of low water river discharge. In October 1936, the upper end of the salt water wedge was located about 22 miles above New Orleans or about 145 miles from the Gulf, at a depth of approximately 120 feet below the water surface of the river.

In locating the wedge, it is constantly observed that the salinity content changes from fairly fresh water, several hundred p.p.m. of chlorine, to dense Gulf water, 10,000 to 15,000 p.p.m. of chlorine, within a depth of a few feet. The line of demarcation between fresh and salt water, referred to as the "interface," is arbitrarily taken as the point at which a chlorine content of 5,000 p.p.m. is observed.

Observations taken during several periods of extreme low water show conclusively that the entrance of salt water into the river is confined to the deep man-made navigable passes, South and Southwest Passes, and that under normal wind and tide conditions no intrusion of dense salt water occurs through any of the other relatively shallow outlets or passes.

Tides - The average daily tidal range at the end of the passes or in the Gulf of Mexico is about 14 to 16 inches, and is about 12 inches at the Head of Passes. During low water a slight tidal effect can be observed in the Mississippi River 35 miles above Baton Rouge.

Wind - Prevailing winds are from the southeast.

ARTIFICIAL FACTORS

Artificial factors affecting sedimentary action in the passes are those improvements or training works which have been performed or constructed in South and Southwest Passes. Regardless of the primary purpose of a project any work which affects the complex regimen of the passes definitely affects the sedimentary action in the delta area.

Jetties have been constructed for the purpose of increasing velocities and inducing greater depths in the bar channel by scour. Spur dikes were installed in the passes and jetty channels to secure lesser channel widths and thereby obtain a more efficient channel hydraulically. Submerged sills were placed across the head of Pass a Loutre and other openings to divert greater river flow through the navigable passes. Outlets in the banks of South and Southwest Passes were opened as required to maintain the integrity of the banks or closed for the purpose of increasing velocities in the jetty channels.

Maintenance dredging is performed annually to provide project depths through the navigable outlets, South and Southwest Passes. Critical areas requiring this corrective action are: Bars which form at and in the vicinity of the Head of Passes and bars which lie just Gulfward of the mouths of the passes that are commonly called "outer bars." As a result of improvement works accomplished to date, South Pass is practically self-maintained, whereas, Southwest Pass requires some dredging in order to maintain navigation depths.

MATERIALS IN TRANSPORT

Investigation of materials-in-transport in the Mississippi River, until recent years, were inadequate in scope to develop reliable scientific procedures. Consequently, the study of the laws governing the transport of sediment by flowing water has been intensified because of the increased rate of construction of river-control works. Transport of bed and soil materials in the Mississippi River and its outlets and the resulting changes in the configuration of the beds are important problems to the designer of projects for the improvement, regulation, and economic development of the river system. The average layman regards the river as merely a natural channel carrying a stream of water, whereas, the engineer has learned that the river is not only a stream of water, but to a greater or lesser degree is also a stream of sediment.

Recognizing the desirability of perfecting methods for measuring the quantity and for determining the character of sediment loads in streams, several agencies of the United States Government organized an Interdepartmental Committee in 1939 to sponsor an exhaustive study of all problems encountered in collecting sediment data and to eventually standardize

accepted methods and equipment. The agencies of the Federal Government which have actively participated in this endeavor include: Corps of Engineers, Department of the Army; Soil Conservation Service of the Department of Agriculture; Geological Survey and Bureau of Reclamation, Interior Department; and the Tennessee Valley Authority. The scope of the general subject "A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams" consisting of several reports, has been completed except for continuing refinements and the procedures established are currently used in connection with sediment studies of the lower Mississippi.

In addition to the general standardization of sediment methods and equipment, the Chief of Engineers recognized the need for intensified research on sediment problems as exists at the mouths of the Mississippi River and at other such coastal areas. Accordingly, the Chief of Engineers on 20 October 1948, established the "Committee on Tidal Hydraulics" with the objectives of: Evaluating the present state of knowledge of tidal phenomena of interest to the Corps of Engineers; recommending programs of study, investigation, and research designed to provide the knowledge necessary to arrive at adequate solutions for the engineering problems associated with such tidal phenomena; and exercising technical supervision of the prosecution of the recommended programs.

This Committee, in fulfillment of the first objective, published Report No. 1 entitled "Evaluation of Present State of Knowledge of Factors Affecting Tidal Hydraulics and Related Phenomena" under date of February 1950 in which the problems of the lower Mississippi River were discussed specifically or by inference.

QUANTITY OF MATERIALS TRANSPORTED THROUGH THE PASSES

In discussing materials transported by the Mississippi the following terms are used as defined:

"Materials in Transport" are all solid materials transported by the river with the exception of dissolved and colloidal solids. Sediment is synonomous in meaning with "materials in transport."

"Materials in suspension" or "suspended load" is that portion of the materials-in-transport which is not in frequent contact with the river bed."

"Bed load" is that portion of the materials-in-transport which is in frequent or direct contact with the river bed."

One of the earliest estimates of the amount of materials transported by the Mississippi through the passes was made by Humphreys and Abbott on the basis of observations taken during the period 1851 to 1853. Their estimate showed the mean amount of sediment discharged annually as equivalent to a prism having a base of one square mile, and a height of 268 feet, or an amount having a weight of approximately 391 million tons.

Captain M. R. Brown, Corps of Engineers, U. S. Army, computed the aggregate amount of suspended sediment discharged through South Pass during the year ending March 25, 1878 to be 23,446,365 cubic yards. The

discharge of South Pass was considered at that time to be approximately 10% of the entire river indicating a total of 234,464,000 cubic yards of suspended material discharged through all passes during the year. **Captain Brown** added 50,536,000 cubic yards for bed load movement along the bottom of the channel, making the total amount of river sediment carried to the Gulf about 285,000,000 cubic yards. This amount of material would cover a one square mile area to a depth of 276 feet, which agrees closely with the Humphreys and Abbot estimate. Using a specific gravity of 1.89 for the sediment average the total amount of material is about 453 million tons.

Results of sediment observations for the period 1879 to 1898 are printed in the annual reports of the Chief of Engineers, U. S. Army for 1886 and 1898. The maximum discharge of sediment per annum was 33,416,795 cubic yards in 1888, the minimum was 11,633,280 cubic yards in 1879, and the mean per annum for the 15-year period was 19,719,928 cubic yards. In the annual report of the Chief of Engineers for the year 1894, the amount of discharge through South Pass is given as approximately 10 percent for the years 1877 to 1881, but as 8 percent only in 1894. Assuming that during the period 1881 to 1894, there was a constant rate of decrease in the percentage of discharge carried by South Pass, the sediment discharge through the three passes aggregated 371,297,700 cubic yards in 1888, and the average annual discharge through the three passes for the 15 years, 1879 to 1893, was about 214,347,000 cubic yards, which was approximately 587,250 cubic yards per day. Using 1.89 as the specific gravity of the transported material, the annual total would be 592 million tons for the year 1888, and an average annual total of 341 million tons for the 15-year period.

There were no further investigations of materials in transport in the passes until early in the year 1938 when an investigation was initiated by the District Engineer, First New Orleans District. From the set of observations obtained in this investigation, the annual discharge of suspended sediment through Southwest Pass, South Pass, and Pass a Loutre was computed to be 514 million tons. At that time approximately 15 percent of the river discharge was going through Cubits Gap and other outlets so that this figure does not represent the total suspended material carried to the Gulf.

Annual discharges of suspended sediment computed from measurements of the Carrollton range at New Orleans during the years 1851, 1852, 1879 to 1880, and 1929 gave amounts of 391 million, 572 million, 363 million, and 845 million tons per year, respectively. The average of these measurements would indicate suspended sediment discharge of approximately 543 million tons annually.

The suspended sediment discharge of the Mississippi River at Baton Rouge, Louisiana, computed from observations for the water year commencing October 1949 was 544 million tons, which compares favorably with sediment discharges computed for the Mississippi River at New Orleans and at the passes.

An examination of the results of sediment observations from Baton Rouge to the Gulf indicates a fairly uniform suspended sediment concentration at stations in this reach. There has been no indication that the bed of the lower Mississippi above the Head of Passes is rising and it

follows that if the bed of the river is not being built up in its lower reaches by deposition, the sedimentary load, however carried, must be deposited over its banks or transported to its mouth. In this case the overflow is prevented by levees and the sedimentary load must be carried to its delta. No sediment of consequence is delivered to the Mississippi River from other streams below the latitude of Old River, 322 miles above its mouth. Therefore, the sediment content of any given volume of water during this 322 mile travel must be fairly constant whether carried as a suspended or bed load. There has been no connected series of sediment observations to confirm such a theory but for practical purposes the sediment discharge measured at Baton Rouge can be considered indicative of the amount carried to the passes.

The "bed load," or unmeasured part of the materials in transport, is estimated to be 25 percent of the measured suspended load. This percentage is higher than is generally estimated for Mississippi River bed load, in that it includes the heavier concentrations of the coarser suspended sediments which pass below the lowest sampling level. On the basis of this estimate and an annual suspended sediment discharge of 544 million tons, the bed load would amount to 136 million tons. The total of 680 million tons is for annual flows comparable to the water year of 1950.

The water year of 1950 was characterized by an unusually long high water period. The annual volume of flow for the 15 water year period from 1936 to 1950, inclusive, averaged about 72 percent of the 1950 discharge volume. Allowing that the sediment discharge would be reduced proportionately, the average amount of sediment transported annually, based on the 1950 measurement, would be approximately 500 million tons.

The sediment discharge through any particular pass can be considered as a function based on the percent of water discharge through that pass to that of the main river. The present discharge through Southwest Pass is about 29 percent; through South Pass about 15 percent; through Pass a Loutre about 37 percent, and through Cubits Gap and other outlets about 19 percent. Assuming an annual 500 million ton sediment load for the river, this would give annual sediment loads of 145 million tons for Southwest Pass; 75 million tons for South Pass; 185 million tons for Pass a Loutre; and 95 million tons for Cubits Gap and the other outlets. The manner in which this material contributes to the sedimentation in the passes and vicinity will be discussed later.

CHARACTERISTICS OF MATERIALS

At the mouth of the river the suspended material usually consists of very fine sand, silt and clay. The finest grades and the lowest degree of concentration are found near the water surface. The coarseness and the silt content increases toward the bottom with the heaviest grades near the bottom. Very coarse sand and gravel are rolled along the bottom.

In the 1938 study of suspended materials transported through the Passes, no samples were found to contain material exceeding the very fine sand particle size. Moreover, only a small percentage of the material fell within the very fine sand range. On the basis of the Udden or modified Wentworth-Udden classification of sediments according to particle size, a typical suspended sediment sample from the 1938 observations would consist of 2 percent

very fine sand, 0.125 m.m. to 0.0625 m.m. in size; 48 percent silt,
0.0625 m.m. to 0.0039 m.m. in size; and about 50 percent clay, 0.0039 m.m.
to 0.0001 m.m. in size. The 1938 mechanical analyses of materials taken
from the river bed and from the passes showed that the largest particles
were retained on a sieve having a mesh opening of 3.327 m.m. and that all
were under 4.669 m.m. The largest size particle contained in most of the
samples was between 1.168 m.m. and 1.651 m.m. All bed samples contained
material graded down to the clay size.

SEDIMENTATION IN THE LOWER MISSISSIPPI RIVER

The Mississippi Delta is a complex mass of sediment deposited by a
series of ever-changing distributaries which carry an enormous amount of
water and sediment to the Gulf of Mexico. The majority of the sediment
consists of silt and clay and is deposited in the Gulf close to the river
mouths. The coarser material, silt, is deposited on the outer bars of the
outlets, chiefly on the west side. Most of the finer, clay-sized particles
remain in suspension and are carried beyond the outer bars where they grad-
ually settle to the Gulf bottom.

For purpose of comparison, the lower Delta is divided into the Pass a
Loutre area which is essentially unaffected by the works of man, and the
South Pass and Southwest Pass areas which have been so modified by jetties
and dredging as to require separate analysis.

Sedimentation in the Pass a Loutre area follows the general laws of
nature and is affected primarily by the type and amount of sediment,
gradient of the stream, action of the Gulf at the location of discharge, and
the effects of the littoral currents. The littoral currents in the vicinity
of the Delta have not been studied in sufficient detail to establish their
effectiveness as transportation agents. It is known, however, that the pre-
vailing winds in this area cause surface circulation which is similar in
effect to a littoral current. The eastern channels face almost into the
prevailing wind, with the result that sediments are spread fanwise across
the entire eastern front of the region. Northeast Pass and Southeast Pass
are almost tied together by a long spit which completely blocks these two
channels to navigation, even by shallow-draft boats.

In comparison, the construction of jetties and dredging operations have
caused a drastic change in the manner and rate of sedimentation in South and
Southwest Passes. These confined channels are much deeper, and, as compared
with the other passes, are essentially static. Captain Eads, by the con-
struction of the South Pass jetties, contracted the mouth of the river, with
the resultant deepening of the channel across the outer bar. His results
were eminently successful except that he did not at first recognize the im-
portance of prevailing currents in causing an excess of deposition on the
central and western side of the channel bar.

Due recognition is now given to the existence of the littoral currents
Gulfward of the passes without which the economic maintenance of the bar chan
nels at South and Southwest Passes would be impossible. There is, however,
some difference of opinion among authorities as to what causes the shoaling
of the bar channels and the source or sources of the deposited material.

During flood stages of the river, the bottom layers of fresh water

discharged by passes carry an excessive load of sand as compared to the surface layers. At South Pass this portion of the flow did not follow the line selected for the channel through the bar. At Southwest Pass, attempts to maintain a channel through the bar were confined to a zone traversed by the most heavily laden part of the fresh water stream, that is, a bar channel on the prolongation of the jetty channel. Efforts to secure such a channel through the bar by dredging at South Pass, extending over a period of 20 years, and similar efforts at Southwest Pass extending over a period of 13 years, were never successful. Deposits during high river stages always exceeded the amount removed by dredging. Bar channels of project dimensions at both passes were not secured until this channel, with respect to the jetty channel, was inclined about 36 degrees to the east. It can be said with respect to both passes that the adoption of lateral channels through the bars marked the turning point from failure to success in maintaining channels of project dimensions.

By far the larger portion of the material reaching the outer slopes of the bars, whether it reached there from suspension, or as a result of being pushed along the bottom, is carried to the west by the littoral current. Even if a small portion of the sediment discharged by the passes during flood stages were to be deposited, and remained in the immediate area in front of the jetties, South and Southwest Passes would soon be closed to navigation.

As previously described, a salt water wedge enters the navigable passes during low river stages. As the fresh water erodes along the plane of the salt water wedge, the deposition of some of the suspended load is hastened by coagulation, or flocculation. Since it is known, from observations during flood stages of the river, that a portion of the suspended load is deposited at least 20 miles from the passes, it follows that precipitation of this loss is gradual and cannot be so rapid as to cause appreciable shoaling in the jetty and bar channels. The coagulated mass of mud is so soft it is usually difficult to detect with a lead line and is usually moved out into the Gulf during early flood stages of the river. It seems certain, therefore, that most of the troublesome shoaling is caused by the bed load of sediment carried down the passes.

Observations at the mouth of Southwest Pass show that at about 800,000 c.f.s. in the Mississippi at New Orleans, the fresh water currents have sufficient force to hold the salt water wedge just outside the jettied channel. Naturally, the wedge would be expected to work in and out of this channel with the flood and ebb of the tide in the Gulf, but the average position is about the ends of the jetties. At the latter point the flood stage shoaling commences with a Mississippi River flow of 800,000 c.f.s. at New Orleans. As the river discharge increases the salt water wedge is forced down the bar channels and is followed by shoaling.

Although the salt water wedge is above the ends of the jetties at flows below 800,000 c.f.s. at New Orleans, no shoaling of consequence occurs although suspended sediment concentrations at this range of flow often exceed that found at higher river discharges. Shoaling follows the salt water down the bar channel and is coincident with the salt water wedge at that point and about 800,000 c.f.s. discharge at Carrollton.

Comparing low water conditions in the bar at South Pass with Southwest Pass, it is found that while the latter bar channel is gradually shoaling, the former is scouring deeper. There are logical reasons for this inverse action in the South Pass bar channel. Some of the features which probably contribute to the favorable result are worthy of consideration. The axis of the South Pass bar channel is in a more favorable direction with reference to the littoral current and prevailing winds. The bearing of the South Pass bar is about S. 65° E.; the corresponding bearing at Southwest Pass is due south, a difference of 65°. Taking the direction of the prevailing winds as due Southeast, or S. 45° E. for a season average, the unfavorable wind angle at South Pass is 20° as compared with 45° at Southwest Pass. The littoral current in its approach to the mouth of South Pass does not traverse the lower reaches of Garden Island Bay as it does the reaches of East Bay in its approach to the mouth of Southwest Pass.

There is a small shoal area to the east and north of the South Pass bar channel similar in shape, extent and position to the like area at the mouth of Southwest Pass over which, in both cases, the littoral currents must pass in their approach to the bar channels. But the relative conditions are more favorable to South Pass to the extent of much flatter slopes, between shoal area and bar channel, for the transportation of sand and the reduced effect of wave action in greater water depths.

It would be unreasonable to contend that no material is carried into the South Pass bar channel, during the low river season, from the two adjacent shoal areas, or bars, but it can safely be said that scouring forces in the jettied and bar channels are sufficient to move not only any such material but considerable quantities of additional material deposited in the two areas by the falling river from the previous flood stage. Here again there must be a constant fight between scouring and depositing forces with the ascendency of the former, over the latter, well maintained throughout the low river season. That the scouring forces are greatest must be attributed to greater channel velocities than exist at Southwest Pass.

At South Pass in 1897, about the time the bar channel was inclined to the east, the 30-foot contour was approximately 2500 feet Gulfward of the jetties. Since 1897, the 30-foot contour has advanced about 4,400 feet and is presently approximately 6,900 feet Gulfward of the end of the jetties. The rate of advance of the 30-foot contour since 1897 has been approximately 85 feet a year. Figure 6 shows the progressive advancement of the 30-foot contour from 1897 to 1950.

Since 1922, the year after the Southwest Pass bar channel was inclined to the east, the 35-foot contour has advanced from a position 5000 feet seaward of the end of the present jetties, on a prolongation of the West Jetty axis to a position 8800 feet seaward, or 3400 feet in 28 years for an average of about 122 feet a year. Before 1922, the rate of advance was approximately 385 feet per year. Figure 7 shows the progressive advancement of the 35-foot contour since 1898.

The crest of both bars will continue to advance into the Gulf, the west side faster than the east side as a result of the east to west littoral current. The time will come when the velocity of the river water
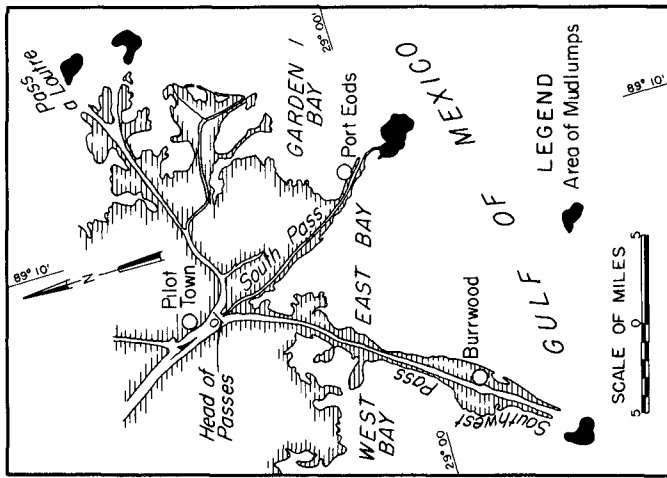
Fig. 8
Location of mudlumps in the
Mississippi River Delta.
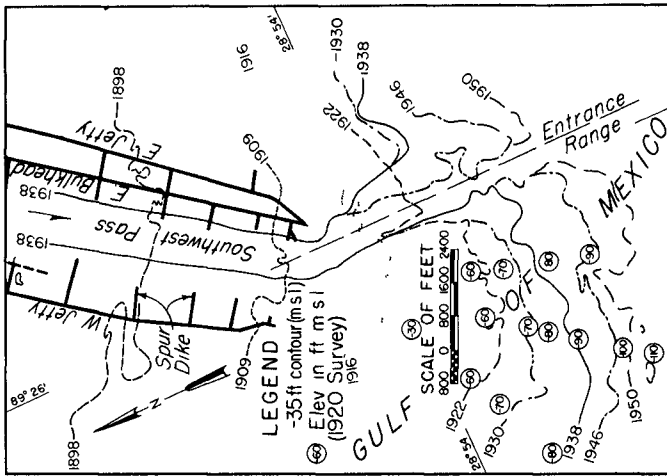
Fig. 7
Advance of the 35 foot below
mean sea level contours, South-
west Pass of the Mississippi
River, 1898 to 1950.
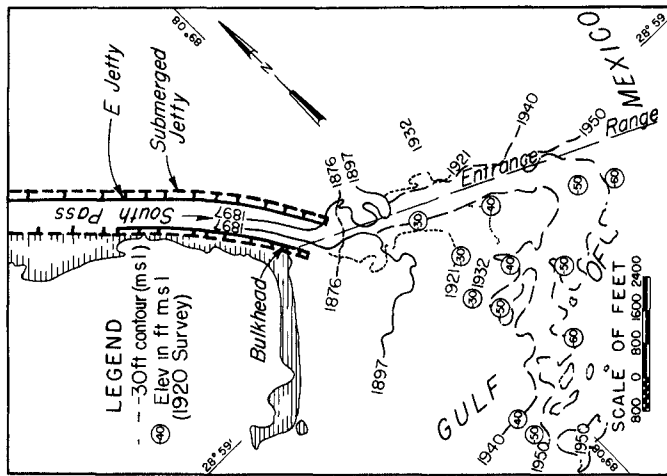
Fig. 6
Advance of the 30 foot below
mean sea level contours, South
Pass of the Mississippi River,
1897 to 1950.

into the Gulf will no longer be sufficient to overcome the resistance across the increased length of the bar. At that time the river will then flow more to the east, this being the path of least resistance. When this occurs, the salt water in the bar channel will be displaced by the silt-laden river water and shoaling will probably follow at a rapid rate. It will then be necessary to extend the jetties. Sedimentation studies of the passes should indicate necessary corrective action before the problem becomes acute.

MUDLUMPS

There is a close relationship between the sedimentary characteristics of the mouths of the Mississippi River passes and the origin and growth of mudlumps in those areas. However, since this phenomena is the subject for another paper to be presented at the Second Conference on Coastal Engineering, discussion of mudlumps will be limited to localities and general observations.

Mudlumps is the term commonly used for the upswelling of clay which forms islands near the mouths of the Mississippi River passes. These features usually are associated with the bars at the outlets of the various passes. At present, the mudlumps are found within a few thousand feet of the mouths of North Pass, Pass a Loutre, Northeast Pass, Southeast Pass, Old Balize Bayou, South Pass, and Southwest Pass. Relative locations of mudlump areas are shown on Figure 8.

Mudlumps appear to be a Mississippi River Delta phenomenon in that they are unknown elsewhere in the world. These islands of mud, because of their striking appearance, constantly changing locations, and proximity to the outlets of the Mississippi River, have caused comment by marine personnel since the earliest days of navigation. In a region having elevation differences of two or three feet the vertical mudlump cliffs of from five to ten feet in height are the most prominent features in existence.

REFERENCES

Committee on Tidal Hydraulics (1950). Evaluation of Present state of knowledge of Factors affecting tidal hydraulics and related phenomena: Report No. 1, Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.

Corps of Engineers (1950). Report upon the Improvement of Rivers and Harbors in the New Orleans, La. District. Extract from the Annual Report of the Chief of Engineers, 1949: Department of the Army, Office of the Chief of Engineers, Washington, D. C.

Elliott, D. O. (1932). The improvement of the Lower Mississippi River for Flood Control and Navigation: Vol. 1, Mississippi River Commission, Corps of Engineers, Vicksburg, Miss.

Federal Inter-Agency River Basin Committee, Sub-Committee on Sedimentation (1948). Measurement of the Sediment Discharge of Streams: Report No. 8, St. Paul District Sub-Office, Corps of Engineers, Hydrologic Laboratory, University of Iowa, Iowa City, Iowa.

First New Orleans District (1939). The Passes, Mississippi River: Brochure, Corps of Engineers, New Orleans, La.

First New Orleans District and U. S. Waterways Experiment Station (1939). Study of Materials in Transport, Passes of the Mississippi River: Tech. Memorandum No. 158-1, Corps of Engineers, Vicksburg, Miss.

Hutchings, Henry, Jr. (1933). Report on Hydraulic Study for Eliminating Dredging in Southwest Pass: First New Orleans District, Corps of Engineers, New Orleans, La.

Lane, E. W. and Borland, W. M. (1950). Estimating Bed Load: Paper presented at the Annual Meeting of the Amer. Geophys Union in 1950. Bureau of Reclamation, U. S. Department of the Interior, Denver, Col.

Morgan, J. P. (1951). The Occurrence and Origin of the Mudlumps at the Mouths of the Mississippi River: Part 1 of unpublished report, New Orleans District, Corps of Engineers, New Orleans, La.

New Orleans District (1941). Report of Improvements of the Passes of the Mississippi River: Corps of Engineers, New Orleans, La.

New Orleans District (1942). Salinity, Mississippi River at and below New Orleans, La.: Review of Reports, Corps of Engineers, New Orleans, La.

Tiffany, J. B., Jr. (1944). Memorandum Report on Proposed Model of Southwest Pass: U. S. Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss. Unpublished report.

U. S. Waterways Experiment Station (1930). Sediment Investigations on the Mississippi River and its Tributaries prior to 1930: Paper H, Corps of Engineers, Vicksburg, Miss.

Chapter 11

MUDLUMPS AT THE MOUTHS OF THE MISSISSIPPI RIVER

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INTRODUCTION

Mudlump is a popular name for the upswellings of clay which common-
ly form islands near the mouths of the Mississippi River passes. These
features usually are associated with the bars at the river mouths. The
bars are localized sedimentary deposits formed where the river waters
enter the relatively still Gulf of Mexico. At present the mudlumps are
found within a few thousand feet of the mouth of North Pass, Pass A
Loutre, Northeast Pass, Southeast Pass, old Balize Bayou, South Pass
and Southwest Pass. Mudlumps appear to be a phenomenon unique to the
Mississippi River Delta for they are unreported from any other locality.

Mudlumps occur either as islands or submarine prominences. The
exposed portions of the island type range in size from small pinnacles
to a maximum area of about twenty acres. The average island studied
has an area of several acres. These island lumps, because of their
striking appearance, constantly changing location, and proximity to the
entrance channels of the river, have excited speculation since the earli-
est days of navigation into the Port of New Orleans. In this region of
low relief, with normal elevations of less than two feet, the vertical
mudlump cliffs five to ten feet high are the most prominent features to
be seen. Adding to an illusion of considerable height, some lumps are
capped with low, conical mud springs or "mud volcanoes". From vents at
the summits of the cones, glistening, highly fluid mud is discharged,
accompanied by varying quantities of gas. Now recognized as secondary
features, these springs for years have been prominently considered in
hypothesizing modes of origin of the mudlumps.

The more numerous submarine mudlumps differ from the island type only
in their position beneath the water surface. For years these submerged
lumps on the bars and near the entrance channels have constituted naviga-
tional hazards. As early as the year 1839 the Corps of Engineers' dredge
Balize spent a month and a half in an attempt to remove a submarine mud-
lump in the mouth of Northeast Pass (Jones, 1841:83).

One of the most significant characteristics of the mudlump islands is
their continually changing size and appearance. The lumps often alter
radically in a few weeks, and sometimes emerge from the water and attain
prominent size within a few days. It has long been noted, and recently
verified through detailed work, that mudlump growth is accelerated during
and immediately after high-river periods. During the high river of 1949-
1950, great changes occurred in the mudlumps of all the important passes.
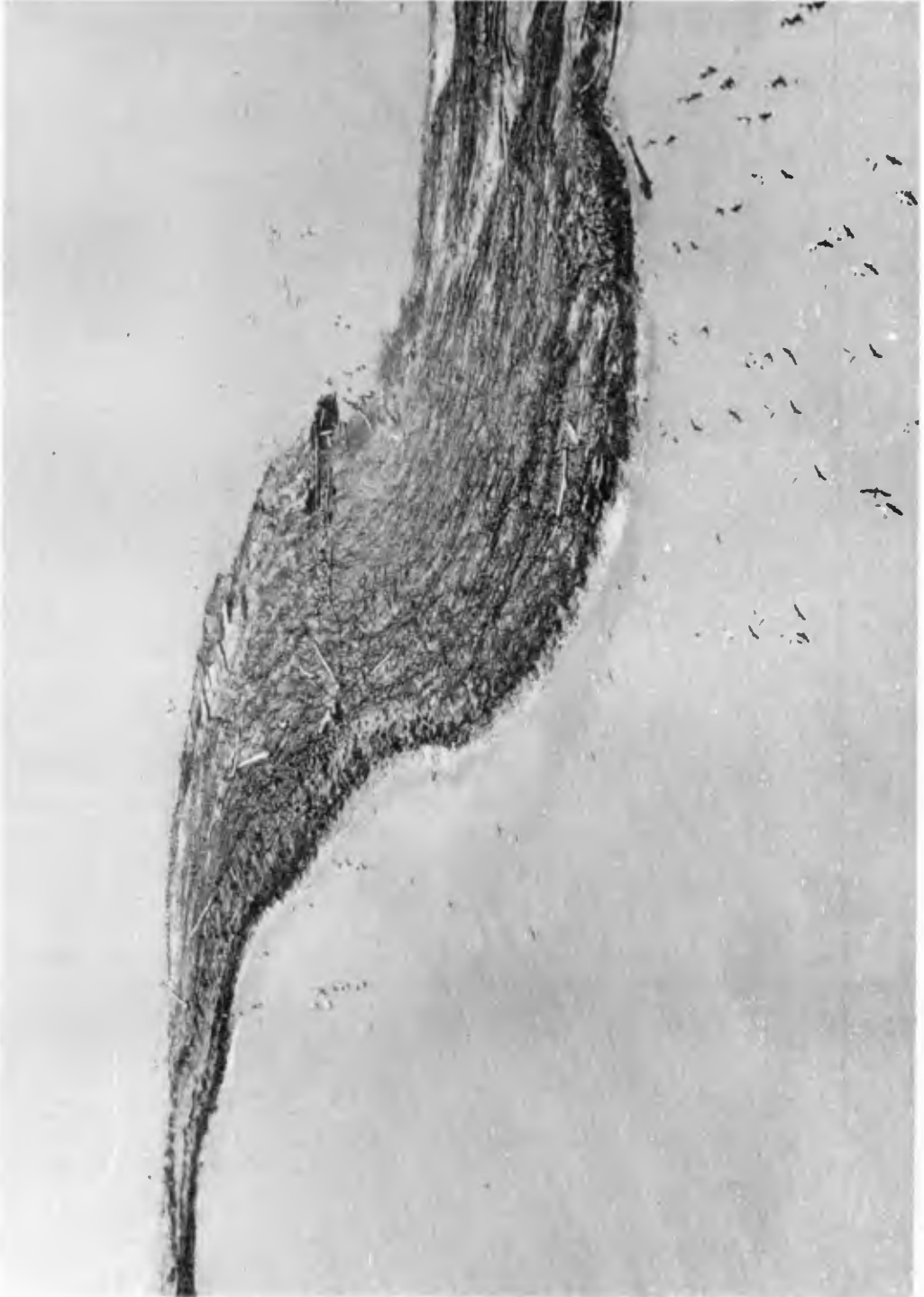
Fig. 1. Aerial photograph of mudlump at the mouth of North Pass showing typical furrowed surface appearance.

The mouth of Pass A Loutre, for example, was carefully studied and mapped at the end of November, 1949. A return trip in April, 1950, revealed that many of the previously noted lumps had been uplifted and enlarged. In addition, two new mudlump islands had been formed, 120 and 250 feet in length, respectively. Both of these new lumps had been uplifted at least eight feet in the intervening four-month period.

The mudlumps are composed of fine-grained sediments, clay predominating. The abundance of clay definitely affects the weathering properties of the island material. At or below sea level, the wet, tenacious clay is practically unerodible. Uplifted into islands, the clay dries, shrinks, cracks, and is readily attacked by rainfall or storm waves. Daily tidal variations allow the Gulf waves to undercut the drying clay, resulting in the formation of cliffs and wave-cut benches. During the summer and fall, uplift of the lumps is negligible, and wave action reduces the area of the high-standing portions. Renewed uplift later elevates the benches, with the result that many mudlumps show raised, wave-cut platforms which can be correlated with periods of mudlump activity.

Many lumps show excellent stratification, with alternating thin layers of clay and silt. Others, non-stratified, are composed of very fine but poorly sorted sediment. The surface of some mudlumps is extremely rough and irregular, with numerous crevasses a foot or more in depth running the length of the island (Fig. 1). This furrowed appearance has been appropriately compared with a freshly plowed field. Most of these fissures are tension cracks, but some are faults having vertical displacement. There is always an escape of gas from these fissures, most noticeable when they are filled with rain water. Inflammable gas in minor quantities is characteristic of mudlump areas, where it escapes from the fissures or the vents of the mud cones.

REGIONAL SEDIMENTATION

There is a close relationship between the sedimentary characteristics of the mouths of the river, and the origin and growth of the mudlumps. The entire Lower Delta is a complex mass of sediment deposited by a series of ever-changing distributaries, which carry an enormous amount of water and sediment to the Gulf of Mexico. The greater part of the sediment consists of silt and clay and is deposited chiefly on the bar in front of the mouth, and laterally on the low or submerged natural levees. Most of the finer-clay-sized particles remain in suspension and are carried beyond the bar where they gradually settle to the bottom.

For purposes of convenience, the Lower Delta may be divided into two parts:

1. The easternmost Pass A Loutre area which is essentially unaffected by the works of man.

2. The South Pass and Southwest Pass areas which have been so modified by jetties and dredging as to necessitate separate consideration.
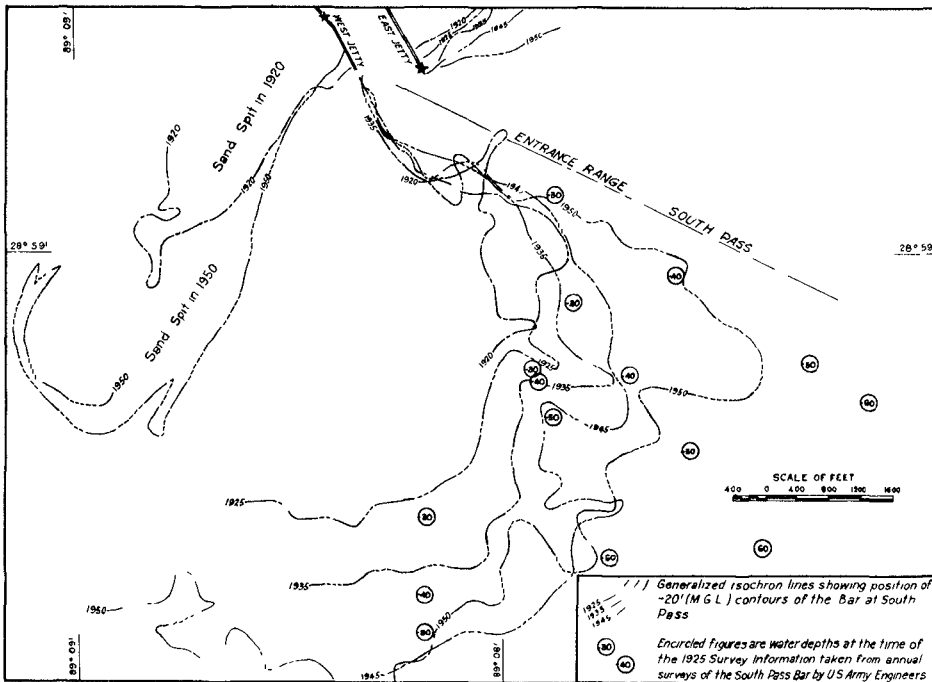
Fig. 2. Advance of the bar at South Pass, 1920 to 1950.
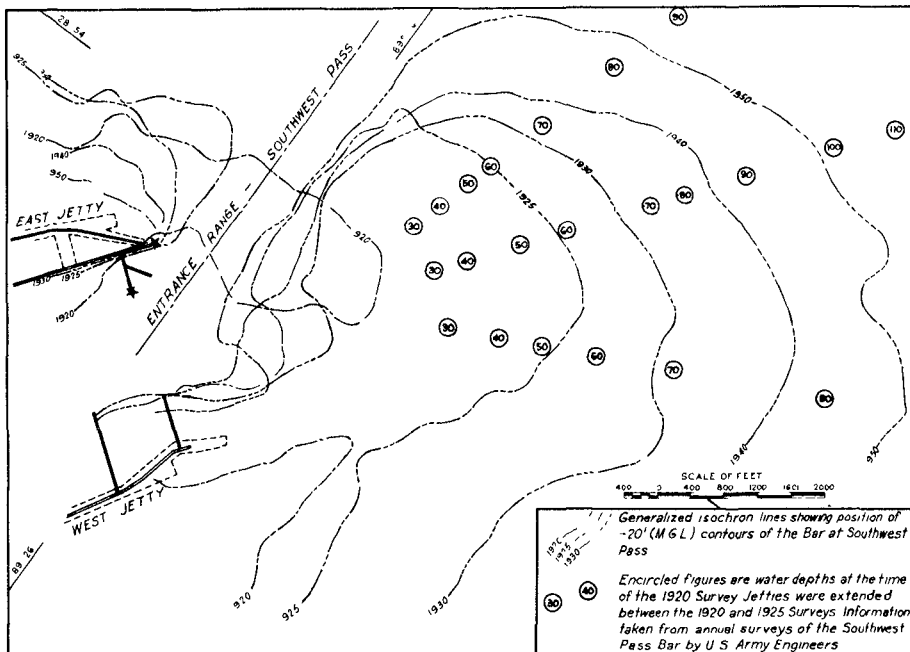
Fig. 3. Advance of the bar at Southwest Pass, 1920 to 1950.

Sedimentation in the Pass A Loutre region follows the general laws of nature. It is affected primarily by the type and amount of sediment, gradient of the stream, character of the Gulf at the position of discharge, and the effects of near-shore currents. The near-shore or littoral currents in the vicinity of the Delta have not been studied in sufficient detail to establish their effectiveness as transporting agents. It is known, however, that the prevailing winds in this area cause surface water circulation which is similar in effect to a littoral current. The eastern channels face almost into the prevailing wind, with the result that sediments are spread fanwise across the entire eastern front of the region. Northeast Pass and Southeast Pass are almost tied together by a long spit which completely blocks these two channels to navigation, even by shallow-draft boats.

On the other hand, the presence of the jetties, supplemented by occasional dredging, has caused a drastic change in manner and result of sedimentation at South and Southwest passes. These confined channels are much deeper and, as compared with the other passes, are essentially static. Captain Eads, by the construction of the South Pass jetties, confined the mouth of the river, with a resultant deepening of the channel across the bar. His results were eminently successful except that he did not recognize the importance of the prevailing wind currents in causing an excess of deposition on the central and western side of the channel bar. By shifting the bearing of the entrance channel twenty degrees to the east, this difficulty has subsequently been overcome. It is important to this study to recognize the fact that at South and Southwest passes, the majority of sedimentary deposition occurs on the western side of the bar. (Figs. 2 and 3). This fact is clearly shown by the development of a long spit or hook extending westward from the end of the west jetty at South Pass. Such a spit has not developed at Southwest Pass for two reasons: First, the angle made by this pass with the prevailing wind is different and the sediment is carried westward into much deeper water. Second, the jetties at Southwest Pass have not been in existence long enough for the surrounding Gulf water to become shoal.

<center>MUDLUMP STRUCTURE</center>

In the course of this investigation, some twenty-five mudlump islands were mapped and studied in detail. For brevity, in discussing their structural nature, a hypothetical example will be employed to typify mudlump characteristics.

The representative mudlump is elongate, the ratio of length to breadth being 3 or 4 to 1, and characteristically is sigmoidal or S-shaped. When exposed at low tide, the wave-cut bench surrounding the island has an area equivalent to or greater than the high-standing portion. Maximum elevations occur along the wave-formed cliffs which flank a lower central area (Fig. 4). The wave-cut bench is composed of alternating thin layers of clay and silt, which dip away from the central part of the island at angles of five to forty-five degrees. The cliffs are composed of stratified sediments which dip slightly, from zero to ten degrees, toward the center of the island. Therefore, these oppositely inclined dips define

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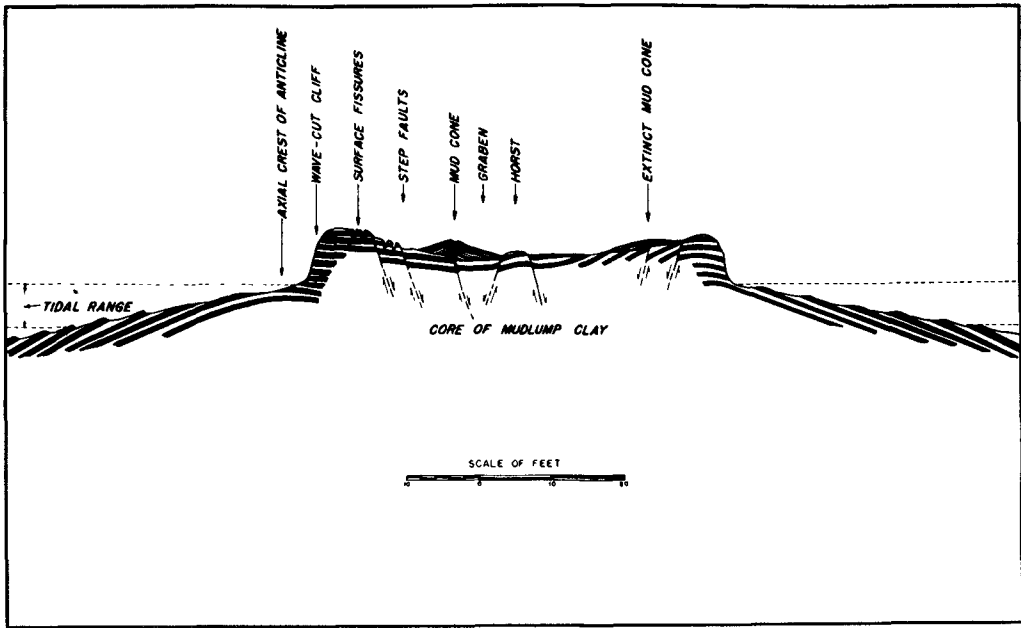
Fig. 4. Schematic cross section of typical mudlump, indicating stratigraphic and structural characteristics.

HYDROMETER ANALYSES
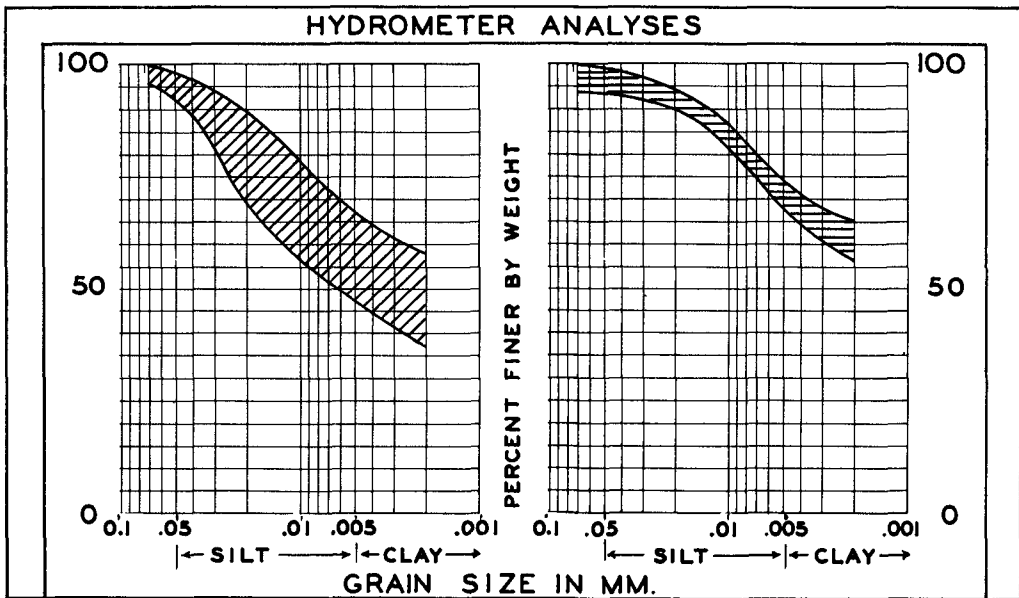
Fig. 5. Grain-size analyses of stratified sediment capping and flanking mudlump islands.

Fig. 6. Grain-size analyses of deep-seated mudlump clay.

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the crest of a low anticline or upfold. This anticlinal crest extends completely around the periphery of the elongate island. The explanation of this peripheral anticline lies, not in the formational upswelling of the mudlump, but in the accompanying sinking of the central part of the island.

As the mudlump material is forced upward, the overlying sedimentary layers are stretched until accumulating tensional forces are sufficient to cause rupture. This rupture takes the form of several normal step-faults generally parallel to the elongate dimension of the island. These faults are usually along one side of the mudlump though occasionally they can be seen on both sides, with the formation of a central graben or down-dropped area. As the mudlump sediments are poorly indurated, the faults commonly develop a braided, branching pattern. The vertical displacement or throw of these faults ranges from a fraction of an inch to over six feet, the average being three to six inches. Within the graben area, the stratified sediments are highly irregular and confused, with the occasional formation of small horsts or upthrust blocks, and the development of numerous tensional fissures. Rainfall, collecting within the lower central area of the lump, often develops small temporary ponds. Rain wash spreads a minor quantity of fine sediment over the floor of the ponds, frequently obscuring the structure of the central part of the island.

Mud or gas vents develop along the faulted periphery of the island, often resulting in the formation of mud cones. Mud is discharged in a broad, thin sheet from the lowest side of the mud-vent rim. Eventually the low side is built up and the direction of flow changes. The first flow then dries, shrinks and forms the typical mud-crack pattern. At a later time this flank of the cone is again covered with thin mud, and the initial mud cracks become filled. In this manner, the entire surface of the cone gradually increases in height with the development of rude stratification. After the cone becomes two or three feet high, hydro-static pressure becomes great enough to cause the mud and gas to break through at another point along the fissure, forming a new vent. Losing its source of supply, the original vent rapidly deteriorates into a low mound. The structural nature of the depressed central area of the mud-lump islands is often difficult to determine because of the masking affect of these mud vent flows.

In considering this hypothetical mudlump, a relatively simple case has been postulated. Field examinations have shown that although most mudlump islands have these common features, their dissimilarities are equally as striking. A major difference concerns their manner of forma-tion. One type appears to have undergone a slow, gradual uplift. This is shown specifically by a low, flat, relatively undisturbed surface. Fissures are few and faults show only minor displacement. Due to the flat domal surface, the area exposed above water varies radically with the fluctuating tides.

In direct contrast is another extreme type of mudlump whose features are indicative of very rapid, violent formation. The entire mass is so

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extensively fissured as to make walking difficult. Many of the fissures
are two to three feet in depth and filled with soft, unconsolidated mud.
On this type lump, the faults often show extreme displacement, in one
case amounting to over six feet of throw. A further diagnostic char-
acteristic is the presence of sheer cliffs both above and below water
level.

With such diverse behaviors during origin, it is natural to expect
variations in the structural characteristics of the islands throughout
their life span.

Another factor which complicates the structure of mudlumps is the
rejuvenation or re-elevation of pre-existing islands. Secondary uplift
of a mudlump seldom is uniform throughout its extent. Because of this
off-centered warping, the island is tilted and deformed, often with re-
sulting secondary faulting. The uplifted wave-cut bench, in such a case,
indicates accurately the amount and location of maximum re-elevation.
Original dips of the stratified sediments are also affected by subsequent
uplifts, often being completely reversed in direction.

MUDLUMP SEDIMENTS

In the course of this study, an attempt has been made to use the
sedimentary characteristics of the constituent material as criteria for
determining the internal structure and method of formation of the mud-
lumps. The results have been only partially satisfactory. Another aim
of the sedimentary study has been to determine the site of deposition of
the fine-grained sediments which ultimately form the mudlumps. This has
been successfully accomplished.

Many mechanical analyses were made of the stratified sediments which
flank and often cap the uplifted mudlumps. The range of these analyses
is shown plotted on Fig. 5. Clay content averages between 45% and 65%,
the remainder being silt with a small percentage of very fine sand. This
should be compared with Fig. 6 which is the range of six samples taken
from a deep bore hole on one South Pass mudlump. Analyzed samples taken
at 20 feet intervals from five to 100 feet indicate that the deeper
material is consistently finer in composition. This constitutes the often
mentioned mudlump clay of the literature. It is very similar to the
stratified mudlump material, but has a higher clay content and proportion-
ately lower silt content.

Mudlumps occur on or near the bars at the mouths of the River passes.
Bar sediments are composed essentially of crudely stratified silt and clay
materials. During periods of high river, the bar increases in size by
the addition of great quantities of fine material. During the ensuing low-
water period, through extended winnowing action, the finer, clay-sized
particles are in great part removed and spread in a thin sheet over the
area seaward from the bar.

Therefore, the resulting bar sediment is a poorly-sorted material,
predominantly silt, with a smaller proportion of clay and still less sand.

Similarly, the sediment seaward from the bar is poorly sorted, but has a relatively higher proportion of clay, with less silt and essentially no sand. The term "pro-delta" clay, as applied by Fisk (1944:59), might apply to these sediments. This finer material, sampled by drag-line several miles out from the river mouths, corresponds closely with the material which has been called mudlump clay. This similarity is corroborated by a separate study of mudlump microfauna made by Andersen (1950). It was found that microfossils incorporated in the South Pass mudlumps almost duplicate those forms currently being trapped in the sediments of the continental shelf beyond the mouths of the passes. Establishment of the deep water origin of these microfossils indicates an uplift of some 350 to 400 feet for the mudlump clay material in which they occur. The stratified sediment which flanks or caps the uplifted mud islands likewise is very similar to the stratified bar material.

MUD VENTS AND GAS VENTS

Mud and gas springs have long been considered an integral part of mudlump activity. Associated with every mudlump mapped during this study have been one or more vents, some discharging mud and building cones, others discharging only gas. The close correlation between vent and lump has caused earlier workers to associate the former with the mode of formation of the latter. Many have postulated that continuous mud flows have built up the island in the same manner that lava flows build up a volcanic island. It has been shown, however, that there is a difference between the stratification of the mudlump body and that of the mud cones which often cap the islands. Furthermore, field studies have indicated that the mud or gas vents most commonly occur along observable faults in the mudlumps. As the faults are primary features formed by the uplift of the mudlump material, the vents in following the fractures must be secondary.

MUDLUMP ORIGIN

Many earlier workers have proposed theories concerning the development of mudlumps. Some of the ideas have been excellent deductions, while others have been only speculations. In order to be acceptable, any proposal made must be in complete accord with all of the available evidence. A list of factual observations must include the following:

    1. Mudlump islands are known to occur only at the mouths of the major river delta.

    2. Mudlump islands occur only at the mouths of those Mississippi River channels discharging into deep water. Mudlump development has never been reported from:

        a. Baptiste Collette Bayou and distributaries.
        b. Cubits Gap distributaries - including Main Pass.
        c. Distributaries from the Crevasse into Garden Island Bay.
        d. Distributaries from the Jump into West Bay.

    These four distributaries all discharge into shallow water.

3. Where mudlumps do occur, they commonly have a right bank location. The one major exception to this rule at the present time is at Pass A Loutre.

4. Both island and submarine mudlumps are found associated with bar deposits at the river mouths (Figs. 7 and 8).

5. Mudlumps develop on the deep-water side of the bar, and retain their position while the extending bar envelops them.

6. The submarine mudlumps in process of development, are shown by surveys to be elongate parallel to the edges of the bar deposits.

7. Bar deposits grow rapidly during river flood stage, and are reduced in size by winnowing action during low river stage.

8. The growth of new island mudlumps may occur as a slow and gradual uplifting process, or it may be a rapid and violent upheaval. The gradual growth occurs during normal seasons, while the violent growth usually follows periods of heavy flood.

9. Secondary uplift of mudlump islands is most pronounced following times of high river stage.

10. Most mudlump islands discharge gas and/or mud through vents or fissures.

11. Mud vents and fissures are genetically related.

12. The discharge of gas and mud is noted to be heavier during times of high river than during other seasons.

The disappearance of mudlumps is a slow process of weathering and wave erosion. The rapidity of the process depends upon the degree of shelter of the mudlump location, the sedimentary composition of the mudlump and the amount of rejuvenation of the island by secondary uplift.

The only conclusion consistent with the factual evidence is that mudlump growth and bar development are complementary processes. The close correlation between increase in bar size and increase in mudlump activity precludes postulating any other mode of origin for these features.

The sequence of events leading to the emergence of a mudlump island is schematically represented in Fig. 9. During Stage A, the river distributary deposits sediments in three transitional zones:

1. A near-shore, massive bar deposit chiefly sandy silt.

2. A thin layer of silty clay grading seaward into

3. a thin layer of plastic clay.

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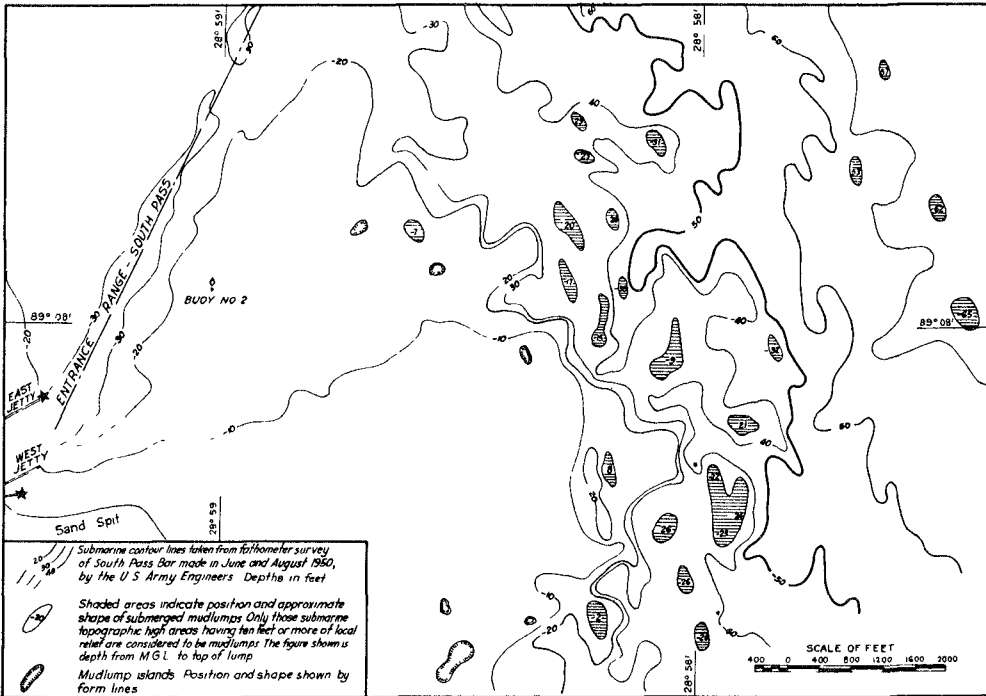
Fig. 7. Map of the bar at South Pass. Submarine mudlumps shown as shaded areas.
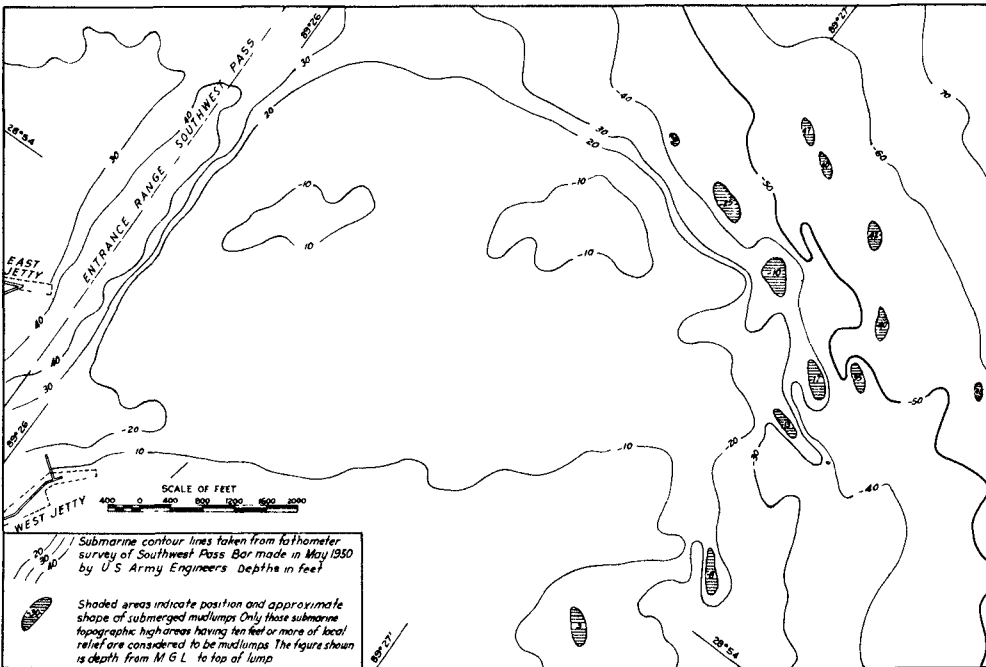
Fig. 8. Map of the bar at Southwest Pass. Submarine mudlumps shown as shaded areas.

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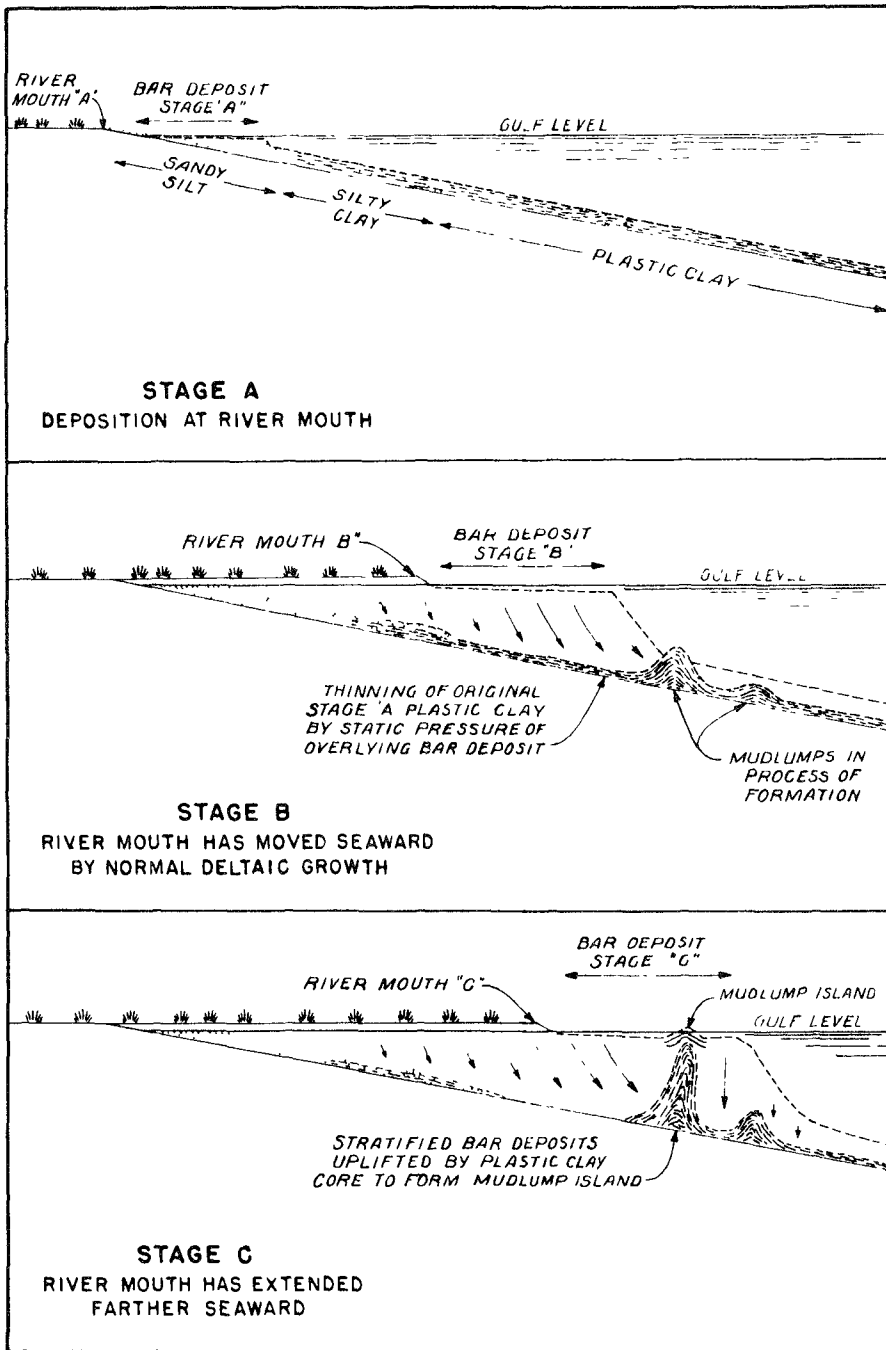
Fig. 9. Schematic diagram showing relationship between bar growth and mudlump development. Vertical scale greatly exaggerated.

As these zones are transitional, both laterally and vertically, they cannot be considered exact. They suffice, however, for purpose of illustration. The distributary continues to extend its mouth, spreading similar deposits and building the bar outward. At Stage B, the river mouth has moved seaward until the silty bar material is overlying the plastic clay layer of Stage A. Due to the static pressure of the bar and its slow forward movement, the plastic clay material is thinned and squeezed ahead. The forward edge of the bar has a relatively steep slope, and at that point the load pressure of overlying sediments is consequently smaller. It is at this place that the plastic clay breaks through to initiate upward mudlump growth. The bar, building forward, surrounds the small, incipient mudlump which becomes stabilized and is forced toward the surface (Stage C). In rising toward the surface, it uplifts the overlying sediments of the bar, which accounts for the stratified appearance of the flanks of many of the lumps. As soon as a mudlump has formed, it becomes localized due to the thick bar deposits around its flanks and the relatively thin deposits over its surface. With increasing static pressure and a sufficient supply of material, the mudlump clay may be forced to the surface.

There is also a good possibility that faulting plays a part in mudlump formation. The thick, localized bar deposits overlying the unstable, newly deposited sediments conceivably could result in a series of faults parallel to the bar front. Such faulting could establish the site of mudlump growth and would serve to break the clay stratum into a series of distinct units. This concept is in accord with the tendency toward the formation of numerous smaller lumps, rather than a few extremely large bodies. However, such an idea does not fall within the realm of factual evidence, and must be considered, at best, as a logical speculation.

Mudlumps are developed only on those passes which discharge into deep water, that is, those channels which approach the edge of the continental shelf. Apparently, only in deep water can the bar deposit become thick enough to exert static pressure of the magnitude necessary for causing flowage within the plastic clay stratum.

CONCLUSIONS

Mudlumps are a Mississippi River phenomenon, unreported from any other delta region. This is due to three important physical characteristics exhibited by the river:

1. The river discharges only fine-grained materials at its mouths. This consists predominantly of silt and clay with some sand.

2. The river discharges into a relatively still body of water. Longshore currents are not strong enough to prevent the formation of a massive bar deposit.

3. The mouths of the Mississippi River are much closer to the edge of the continental shelf than are most other major

delta-building rivers. The resulting steeper slope is favorable for the formation of a thick river-mouth bar.

These three conditions, occurring simultaneously, cause mudlump islands to develop. To the best of my knowledge, the Mississippi River has the only delta which exhibits all of these characteristics simultaneously.

Future mudlump activity at the mouths of the various passes tentatively can be predicted.

1. Southwest Pass will be the site of increased mudlump activity in the future. It will be concentrated on the west side of the entrance channel directly in front of the mouth of the pass. Because of the angle of divergence of the entrance channel, deep water will be east of the mudlump area. It is obvious that dredging must be continued at this pass in order to keep a channel across the bar.

2. South Pass has many mudlump islands at the present time and will continue to develop new lumps in the future. Activity will gradually move south and west of the present area of mudlumps. It can be stated with considerable assurance that mudlumps will not affect the South Pass entrance channel. However, it is probable that the jetties will ultimately have to be extended at this pass. The bar has reached tremendous proportions with respect to the volume of water discharged. It is rapidly encroaching upon the entrance channel and dredging soon may be necessitated.

3. The Pass A Loutre distributaries, Southeast and Northeast Passes and old Balize Bayou, have been affected in the past by mudlump activity but at the present time are almost completely blocked with sediment. It is unlikely that mudlump activity will be evident in these areas because the diminishing load of sediment will cause less static pressure. The mudlumps present in these areas are extremely old and are being incorporated into the marshland at the north and being eroded by wave action at the south.

4. The Pass A Loutre mouth has had a long and erratic history and has been affected continuously by mudlumps. It is hazardous to generalize because of the lack of factual information. Mudlump activity in 1876 closed Pass A Loutre to deep-draft ships. The 1950 mudlump activity necessitated marking a channel for fishing vessels among the numerous islands. It is likely that mudlumps will always affect this pass.

5. North Pass is a comparatively new distributary of the Pass A Loutre system. Its volume of water and sediment is increasing at the expense of Pass A Loutre and the other minor

distributaries. At the present time it is the only channel
of this system which is open to navigation by moderate-
draft vessels. As its sedimentary load increases, the re-
sulting mudlump activity will also increase. It will pro-
bably follow the same pattern of development that has been
exemplified by Pass A Loutre.

REFERENCES

Andersen, H. V. (1950). Recent Foraminiferal Faunules from the Louisiana
    Gulf Coast: Dissertation, Louisiana State University.

Fisk, H. N. (1944). Geological Investigation of the Alluvial Valley of
    the Lower Mississippi River: War Department, Corps of Engineers.
    Published for the Mississippi River Commission, Vicksburg, Miss.
    78 pp and plates.

Jones, A. C. (1841). On the Removal of Obstructions at the Mouth of the
    Mississippi: Journal Franklin Inst., 3 ser., 2:83.

Morgan, J. P. (1951). Mudlumps at the Mouths of the Mississippi River:
    Dissertation, Louisiana State University, 128 pp and plates.

Chapter 12

STRENGTH OF SEDIMENTS IN THE GULF OF MEXICO

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INTRODUCTION

The strength of the sediments in the Gulf of Mexico concerns the
engineer in planning drilling operations. The length and number of
piles to support drilling platforms depends upon the strength of the
mud. Likewise, the size and type of anchors used in securing service
barges is influenced by the strength of the sediments. In the early
stages of developing the offshore oil resources in a new area, the
engineer is handicapped by lack of information as to the nature of the
sediments that must support his structures. As time goes on, he can
base his decisions more and more on experience gained in previous con-
struction operations in or near the area with which he is concerned.
He gradually acquires a mass of data on pile loading tests, settlement
and deformation of existing structures, and laboratory analyses of un-
disturbed samples. He then can design his structure with confidence.
However, in the early stages of the drilling program in any large off-
shore region, such as the Gulf of Mexico, he is confronted with uncer-
tainties, many of which are difficult to resolve because of lack of
definite information upon the strength of the muds.

The geologist can help the engineer greatly in this phase of devel-
opment of the area, because the strength of the sediments depends upon
their geologic characteristics. Basically the strength is determined
by the composition and geologic history of the sediments. Four factors
are fundamental: (1) size distribution of particles, (2) mineralogical
composition, particularly the type of clay minerals, (3) the imposed
load, and (4) the length of time the load has been applied. Each of
these factors is an essentially independent variable, though under
certain circumstances they may be related to one another to a minor
extent. For example, the rate of growth of new clay minerals might
vary with size of particles, the load, or the length of time. Other
factors, of course, influence the strength, but ordinarily to a much
less extent than the four variables just mentioned. Among those that
might be considered are: (1) desiccation, (2) shape of particles,
(3) cementation, (4) circulating water, (5) base exchange, (6) biological
activity, (7) hydrogen-ion concentration, (8) temperature, (9) seasonal
variations in temperature, (10) composition of interstitial water,
(11) reducing potential, (12) earthquake vibrations, (13) diastrophic
deformation, (14) tidal range, (15) barometric variations, and (16) ero-
sional history. Several of these factors are not completely independent
in their effect, as they are related to the four principle variables,
and also to one another. As a rule they exert only a minor effect on
strength; but at times some of them, as is discussed below, may affect

the strength significantly. However, in the present analysis of the problem, the four principal factors listed above have by far the greatest engineering significance.

The relationship between strength and the four primary variables has not yet been worked out, but progress has been made. The rapid development of soil mechanics, under the leadership of Terzaghi and Casagrande, has helped very materially in understanding the relationship between strength and the properties of sediments. Trask and Rolston (1950, 1951) have been endeavoring to approach the problem geologically. Since the geologic nature and history of sediments vary from region to region, empirical relations between strength and nature of sediment in any one area cannot necessarily be applied to another area. In any event, in order to obtain effective interpretation of strength from geologic information, specific engineering data on strength must be correlated with geologic data, but where relationships between strength and nature of sediment have been developed for one region, those data can be applied throughout that area according to the principles of geologic correlation; because sediment of similar geologic origin and similar geologic history will have much the same engineering characteristics throughout the region under consideration. The problem of the geologist is to determine the variations of the properties of the sediment from place to place and to ascertain how they affect the strength.

MAJOR FACTORS INFLUENCING STRENGTH

The most fundamental property influencing the strength is the grain size distribution. A sediment is composed of a mixture of particles of different sizes. The average size of particles differs from one sediment to another. If the particles are coarse, the sediment is a sand, if they are fine, it is a silt or a clay. The average grain diameter is the most useful single figure in describing a sediment. This diameter is called the median, D_{50}, and represents the diameter that divides the sediment into two equal halves by weight with respect to size. One-half the weight of the particles is composed of particles larger than the median, and one-half is composed of particles smaller than the median. As mentioned below, most of the fundamental characteristics of sediments, such as water content, grain size, and shear strength are directly related to the average grain size. For further details of grain size characteristics, see Krumbein & Pettijohn (1938, pp. 228-267).

The strength properties, however, vary with the distribution of particle size on either side of the median. For example, if all the particles are of nearly the same size as the median; that is, if the sample is well sorted according to the geologist, or poorly graded according to the engineer, it is likely to be weaker, because the grains to not interlock as effectively as when they vary widely with respect to size and are well graded. Similarly the particles may be more concentrated with respect to size on one side of the median than on the other. That is, the size distribution may be skewed. The extent of skewness affects the strength properties significantly. For example,

if a sediment composed mainly of clay particles also contains consider-
able sand, it ordinarily is appreciably stronger than if it contains no
sand. On the other hand, if a moist sediment composed largely of sand
also contains considerable clay, it is likely to be weaker than if it
consists entirely of sand. Ordinarily, sediments are described in terms
of the three parameters - median, sorting, and skewness, but in this
paper attention is focussed on the median, as it is the dominant factor
affecting strength.

The water content, W/S, the ratio of the weight of the water (W)
to the weight of the solid constituent (S), is generally recognized to
be an index of the strength of the sediments (Terzaghi and Peck, p.27).
The greater the water content, the weaker is the sediment. The water
content is not an absolute measure of strength, because all sediments
of the same water content do not have the same strength. At least
three factors influence this relationship: (1) the type of clay mineral,
(2), variations in size distribution with respect to the median, and (3)
desiccation, or drying, of the sediment.

Basically the water content varies with the average grain size.
Fine sediments contain much water, coarse sediments, little water. The
forces between the water molecules and the surface of the mineral grains
they wet seem to be the principle factors involved. The total surface
area of the particles in a fine sediment is greater than in a coarse
sediment. Every particle is surrounded by layers of water molecules,
each with its own forces of attraction and repulsion. The zone of
molecular influence extends to an appreciable distance beyond the surface
of the solid particles with the result that there is a minimum distance
to which the particles can approach one another. Thus other things being
equal a high water content is found when the number of particles per unit
volume is high. Also, the attraction for water varies from one type of
clay mineral to another; especially with respect to concentration of
cations. For example a sodium clay has a different affinity for water
than a hydrogen clay (Kelley, 1948). For given size distribution, but
different clay minerals, the water content, and hence the strength, will
vary. The clay minerals also affect the water content in another way,
in that water enters the lattice cells of the different clay minerals
with resulting different effect on the clay. The clay mineral montmor-
illonite is particularly susceptible to water and hence when present in
sediments is an element of weakness.

The relationship between the water content and average grain size
of freshly deposited sediment is illustrated by the upper curve on Figure 1.
The data for this curve were obtained by carefully sorting the particles
of a given sediment into a series of size groups by means of a centrifuge
so that the diameter of the largest particle in each size group was twice
the smallest diameter. The different size groups, obtained in this way
were shaken in water, allowed to settle, and the water content determined.
As can be seen from the graph, the moisture increases in a regular manner
as the grain size diminishes.

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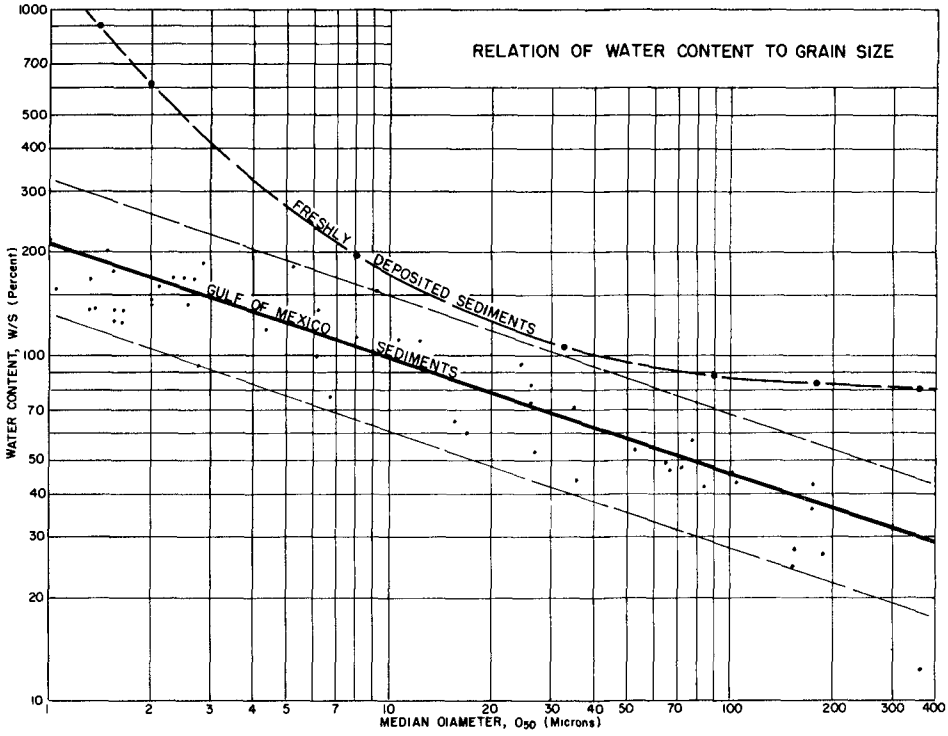
Fig. 1
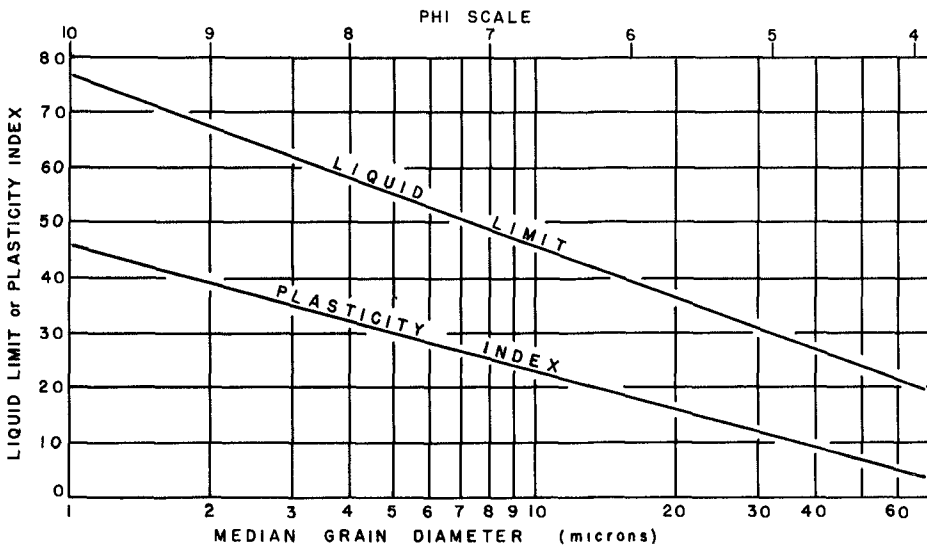
Fig. 2. Relation of Atterberg limits to grain size.

The relationship holds for the sediment in the Gulf of Mexico, as shown by the lower curve in Figure 1. The two curves shown on this figure have different shapes, because the data for the Gulf of Mexico sediments are based on the median diameter of the entire aggregate of particles in each sample, whereas the other curve is based on the mean of small individual size groups. The data for the lower curve are presented without regard to sorting or skewness, yet they demonstrate a definite relationship between water content and grain size, expressed by the formula: $\log_{10} W^3 = 1 - \log_{10} D_{50}$ where W is expressed as a ratio, and D_{50} in microns. The samples on which this curve in Figure 1 are based were collected by an expedition to the Gulf of Mexico sponsored by Woods Hole Oceanographic Institute and the Geological Society of America. (Trask, Phleger, and Stetson, 1947)

The water content, as mentioned above, is influenced by the type of clay mineral. This relationship is obvious from Casagrande's (1936) classic study of the relation between plasticity index and liquid limit. For descriptions of these properties, see Terzaghi & Peck (1948 p. 32). Casagrande shows that sediments in individual geologic environments have consistent relationships between these two Atterberg properties, but that the relationships may differ from one region to another. The relationship in any particular area is materially influenced by the grain size, as is illustrated by Figure 2, based on sediments in San Francisco Bay. As the Atterberg limits are measured in terms of water content, and as the water content is influenced by the grain size, they also should vary with the grain size. The Atterberg limits in Figure 2 when plotted according to the procedure of Casagrande, lie within a narrow band just above his A line, as is illustrated by Figure 3. This band corresponds to the position of his glacial clays.

If the clay minerals are of one type, as they almost certainly must be in a small area such as San Francisco Bay, their effect on the water content should be similar from one sample to another, with the result that the relationship between Atterberg limits and grain size or between water content and grain size should be a simple linear function, but if the type of clay mineral should differ from one sediment to another, the effect of the different kinds of clay on the water content would cause appreciable deviation from the simple linear relationship between water content and grain size. It could thus be postulated that if the sediments in an individual region exhibit a consistent linear relationship between Atterberg limits and grain size, the clay minerals are similar throughout that region and if the relationship is not consistent the clay minerals are not the same throughout the region. Further research on this hypothesis might lead to useful conclusions in interpreting Atterberg tests.

The strength also depends upon the load and the length of time the load has been applied. Sediments when first deposited have the maximum possible water content. As time goes on following their burial beneath later deposits, the particles are pressed closer together, smaller grains find their way between larger grains, clay minerals grow in size, new minerals are formed, and the clay minerals flatten out. All these processes cause the water content to decrease and the strength to increase, but not necessarily to the same degree.

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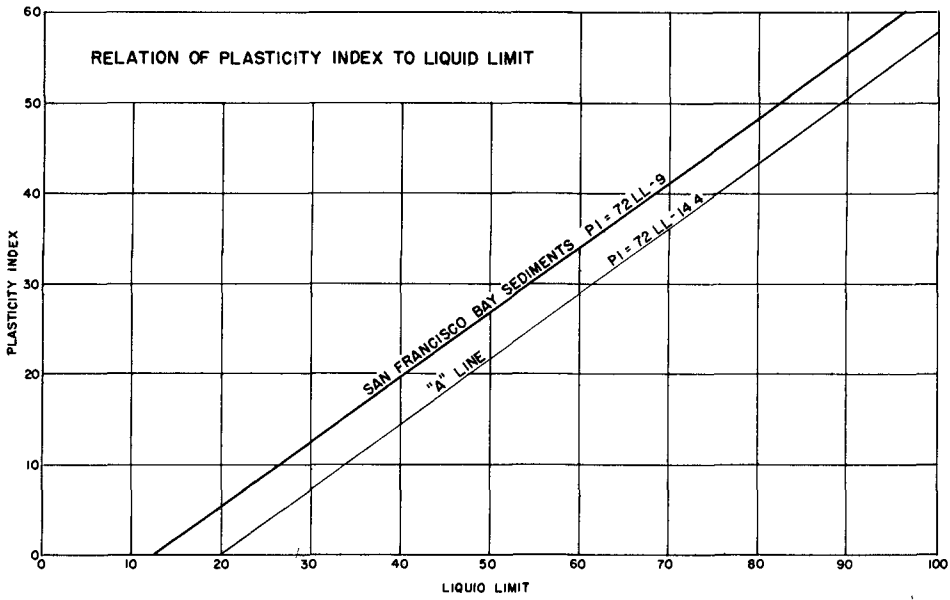
Fig. 3.
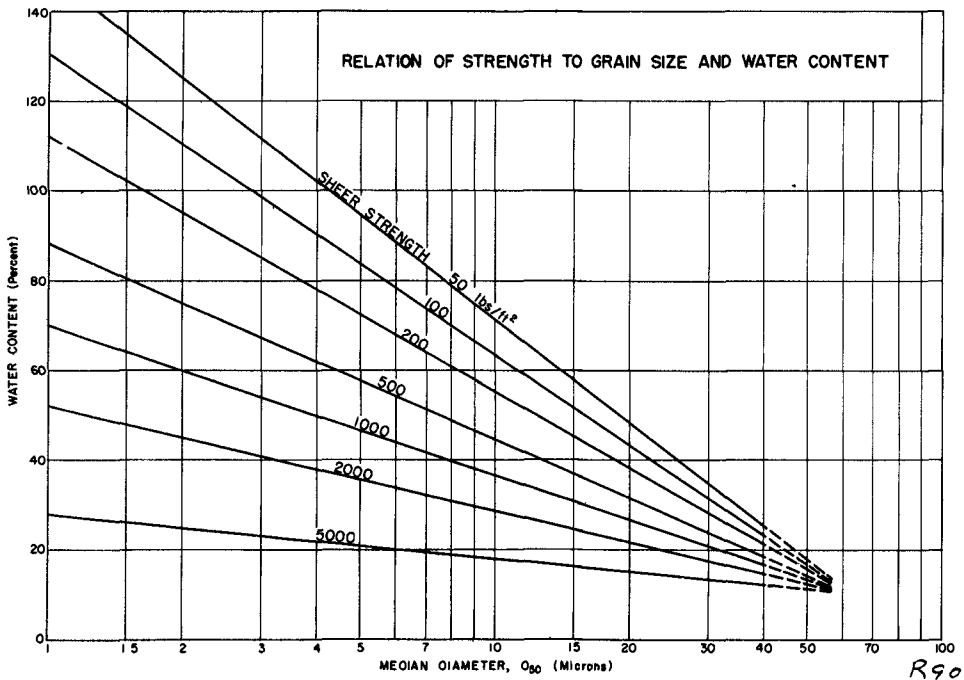
Fig. 4.

Soil mechanics engineers in making consolidation tests have found that when a load is imposed upon a sediment, the water content diminishes at a uniform rate with respect to geometric increases in load once a load has been added equivalent to the normal load that overlies the sediments at the time the sample is collected. The normal rate of increase is called the compression index (Terzaghi & Peck, 1948 p. 65). It is remarkably constant in most fine-grained sediments until the volume has been reduced to within 10 or 15 percent of the maximum possible consolidation. The last 10 or 15 percent of the consolidation, called secondary consolidation, progresses at a very slow rate and is akin to what the geologist calls geologic compaction. It does not affect the present discussion.

Freshly deposited sediments, burdened with essentially no load, could be regarded as unconsolidated. The flat part of their consolidation curves, accordingly would be small. Practically all of the consolidation curve would consist of the steep or virgin part.

This phenomenon is clearly indicated by the sediments in the Gulf of Mexico, which no matter whether fine sand, silt, or clay lose approximately 30 percent of the water in the first 12 to 20 inches of burial. This conclusion is based on a study of the moisture content of a series of cores collected on the continental shelf of Texas and Western Louisiana. (Trask, Phleger, and Stetson, 1947) Though the rate of loss with respect to depth or load in the upper layers of sediments is the same for differing grain size, the water content itself varies with grain size as shown on Figure 1. It is low in the coarse sediments and high in the fine deposits.

The rate of loss of water also is influenced by the permeability. The greater the permeability the greater is the rate of loss. Other things being equal, permeability varies with grain size and with sorting. (Krumbein & Monk, 1942) The larger the grain size the more permeable are the sediments. Likewise, the more nearly equal the particles are in size, for given grain diameter, the more permeable is the sediment. If sediments are coarse, the pores between particles are large, and water can readily escape. If particles are all of the same size the spaces between particles are not filled with smaller particles and thus the pores are relatively large. On the other hand, poorly sorted deposits and sediments with high skewness will consolidate slowly. Ordinarily, the rate of deposition of sediments is so slow that the water content will continually be in equilibrium with the imposed load. This relationship is attested by results of consolidation tests which show that the consolidation load, Pc, is essentially equivalent to the normal load. Thus if the sediments are similar in character with depth of burial, the water content of sediments at given depth could be predicted upon the basis of the compression index. If the relation of compression index to grain size distribution is known, water content at any depth could be inferred from size distribution. A preliminary analysis of some 300 consolidation curves in San Francisco Bay shows a fairly definite relation between compression index and grain size. The finer the grain size the higher

the compression index. The degree of sorting and skewness of the sediments are complicating factors, and in the present stage of analysis of the results, definite generalizations cannot be given. The uniform rate of decrease in water content in the surface sediments in the Gulf of Mexico suggest that perhaps sorting and skewness are more effective factors in consolidation of these sediments than size of grain. Obviously this discussion of consolidation does not apply to sands, which cannot consolidate further after the sand grains come to rest upon one another in the position of minimum porosity.

The time factor becomes important in areas where there has been diastrophic activity, or where sea level has been lowered during some glacial epoch (Umbgrove, 1930). Under these conditions the sediments possibly may have been exposed to erosion, desiccation, or changing water content. Subsequently they may become buried beneath other deposits. The strength of the buried sediments, therefore, would depend upon the previous geologic history and would not be a simple function of depth of burial. This condition is of concern to the engineer in his concept of "preconsolidation" (Terzaghi & Peck, 1948, pp. 67-73). Since the evidence for a widespread lowering of sea level during the glacial epoch, with ensuing period of erosion and exposure, is constantly increasing, it follows that sediments near rivers and harbors along the coast have been subject to erosion or desiccation. Glacial lowering of sea level would be worldwide. Thus engineers should consider the possibility of encountering preconsolidated sediments at shallow depth, when dealing with problems of coastal engineering. On the Gulf Coast, as Russell (1952) in his paper for this Second Conference on Coastal Engineering has shown, the Pleistocene Lissie formation has been found beneath younger deposits. Accordingly, it may be encountered within shallow depth near shore.

For given grain size, strength of sediment varies with the water content. This phenomenon is characteristic of consolidation, because, as individual sediments consolidate under increased loads the water decreases. The load thus is carried to a progressively greater extent by the solid constituents. That is, more of the solid particles press against one another and the greater is the resulting strength. In San Francisco Bay the strength of the sediment has been found to vary with the grain size and water content (Trask & Rolston, 1950). The relationship is indicated by the equation:

$$S_s = \text{Log}_{10}^{-1}\left(\frac{7.5 - 3W - 4\text{Log}_{10}D_{50}}{1.8 - \text{Log}_{10}D_{50}}\right)$$

where S_s is the shear strength expressed in pounds per square foot (one-half the unconfined compression $Qu/2$), W is the moisture content expressed as the ratio of weight of water to weight of solid particles; and D_{50} is the median diameter in microns. (This equation is presented graphically in Figure 4.)

The relationship found in San Francisco Bay cannot be accepted as any general principle until other areas have been studied in like manner.

Since the water content depends to some extent on the type of clay minerals, and since the strength is influenced by the sorting and skewness of the constituent particles, the numerical relationship expressed graphically in Figure 4 may differ in different regions. However it is possible that a similar relationship exists in almost any area where the clay minerals are not too variable in composition.

As applied to the Gulf of Mexico, the strength of sediments at a depth of one foot beneath the surface would average 150 lbs. per square foot and range almost entirely between 50 and 250 pounds per square foot. Data from consolidation tests would materially help the prediction of strength at greater depths.

MINOR FACTORS

The minor factors that influence strength need be discussed but briefly. As a rule they affect the strength relatively little. The shape of the particles is possibly the most important minor factor, though conceivably desiccation locally may be more significant. If sand grains are well rounded they do not wedge into one another effectively; hence the sand is less stable than if the sand is composed of angular constituents. Similarly the platelike clay minerals conceivably could form planes of weakness upon which the sediment might slip under tangential stress. Also, the different clay minerals may have different affinities for water and hence different strength relationships. (Kelley, 1948).

In general the effect of mineral constituents, other than clay minerals may be assumed to be minor, as one mineral differs very little from another mineral as a means of imparting physical strength. Except for clay minerals, the great bulk of the constituent particles of sediment are composed of quartz and feldspar, which are similar in character. Minor minerals such as mica, zircon, or hornblende might have characteristic shapes which compared with other minerals might impart additional strength or weakness to sediment, but only to a minor degree. Mica possibly might be most effective in this respect. It consists of flat plates that provide horizontal planes of weakness.

In the course of geologic time old minerals grow in size and new minerals form, thus imparting strength. Some of the newly formed minerals are deposited in such abundance as to form cement. This cement influences considerably the rate at which water circulates through the sediments. Base exchange affects the properties of clay minerals. In the course of time, it even may result in the formation of clay minerals of a different type and different affinity for water than originally present in the sediments, with resulting change in strength. Biological activity is restricted mainly to the upper layers of sediments, though bacteria can be expected at any depth. The by-products of these living organisms conceivably could affect base exchange, hydrogen ion concentration, and reducing potential, but as the effect of living organisms is

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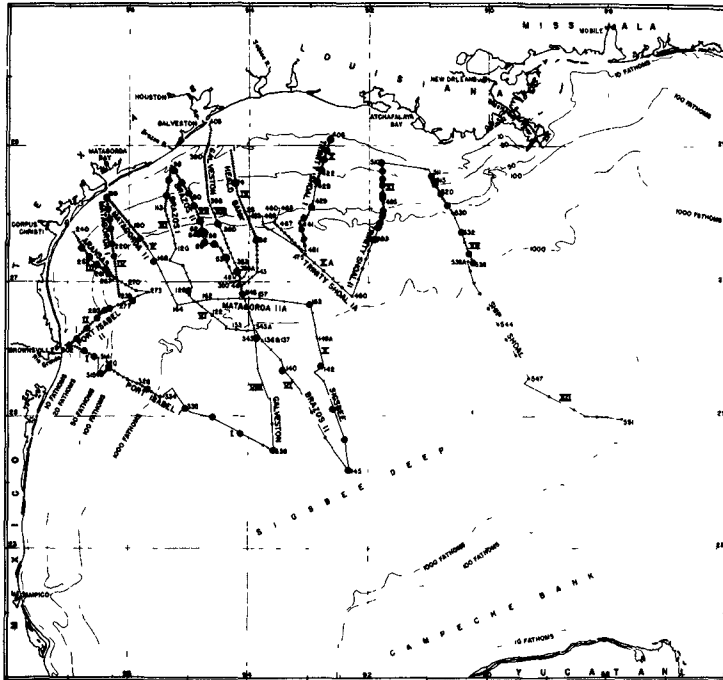
GULF OF MEXICO, WESTERN PART
STATIONS OCCUPIED DURING ATLANTIS CRUISE, 1947
● - WATER CONTENT SAMPLES
SPONSORED BY WOODS HOLE OCEANOGRAPHIC INSTITUTION
AND GEOLOGICAL SOCIETY OF AMERICA

Fig. 5.
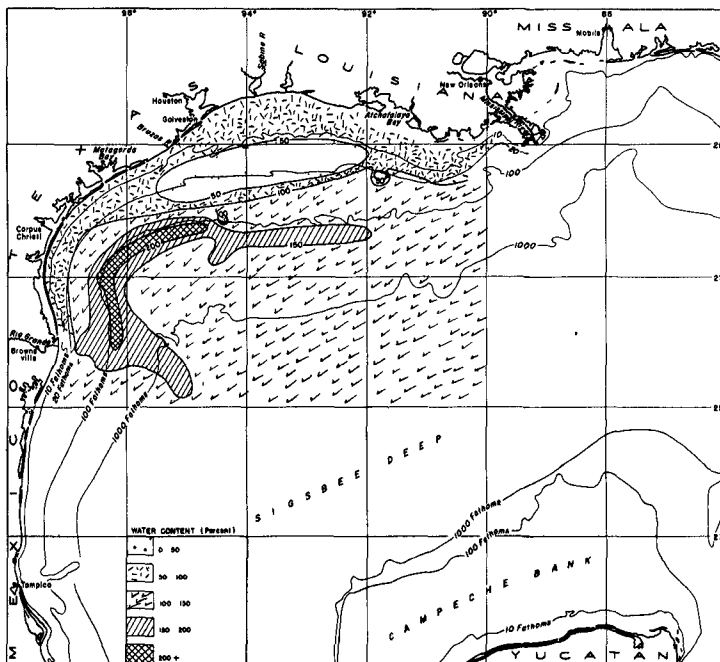
Fig. 6. Water content of sediments, Gulf of Mexico.

largely restricted to the upper layers of sediment, the effect of
organisms on strength ordinarily is small. Continued ingestion of
sediments by worms and other bottom living organisms perhaps may affect
the ultimate strength characteristics of the sediments, particularly
fine-grained sediments. The hydrogen-ion concentration, reducing
potential, temperature, variations in temperature, chemical content
of contained water influence strength properties, such as cementation,
base exchange, and growth of clay minerals; but the overall effect on
strength as a rule presumably is small. Earthquake vibrations might
cause sands to settle and can break up bonds between clay minerals that
have started to grow together, thus weakening the sediments. The effects
of diastrophic deformation and erosional history have already been dis-
cussed. Tides, by exposing sediments alternately to water and to air
and also by changing the pressure to a more or less slight extent upon
the underlying sediment, might exert a minor influence on the strength
of sediments. Barometric pressure could act similar to tides. Also,
microseisms caused by intensive barometric disturbances perhaps might
lessen bonds between mineral constituents.

SPECIFIC CONDITIONS IN THE GULF OF MEXICO

This paper is written without benefit of engineering data on the
sediments of the Gulf of Mexico. Obviously, the geologic conditions
must be tied in with engineering data before they can be used effectively
in predicting strength of sediments in any new area such as the Gulf of
Mexico. The sediments in this area in general are soft. The water con-
tent as indicated by a series of samples collected by the Atlantis
expedition to the Gulf of Mexico in 1947 (Trask, Phleger & Stetson, 1947),
are soft. The location of the samples collected is given in Figure 5, and
the areal distribution of water content in Figure 6.

The sediments lying between depths of 10 and 20 fathoms in an area
bordered on the west by the Brazos River and on the east by the Atchala-
faya River are much coarser and have a lower water content than deposits
in adjacent waters. The strength of the sediments in this central area,
therefore, should be greater than in other areas. About 15 of the samples
taken, indicated by small dots in Figure 5, were cores ranging up to 10
feet in length. The water content of long cores was not determined, be-
cause there was no practicable means of measuring the water content
accurately without interfering seriously with other studies that were
planned for these cores. The water content was determined on short cores
specially collected for such analyses. These cores range in length up to
20 inches. They were hermetically sealed as soon as collected and analyzed
shortly thereafter, in order to minimize possible loss of water through
the seals at the ends of the tubes.

As mentioned above, the moisture content of the sediments at the
bottom of the tube was approximately 30 percent less than in the upper
part. The loss was essentially the same for sand, silt, or clay. The
texture of the sediment at the bottom of most of the cores collected
for water analyses was approximately the same as at the surface. In a
few samples where the bottom of the core was coarser than the top, the

water content was correspondingly lower, but in others, the upper part was coarser than the bottom part, and the moisture content was correspondingly higher. With respect to the overall average loss of 30 percent water, the effect of these two differences largely counterbalanced each other.

CONCLUSION

The argument as developed in this paper has been largely theoretical, because of the scarcity of data correlating the geologic characteristics of the sediment with the strength for engineering purposes. With what is now known about the relation of strength to grain size, and if the general distribution of grain size and water content, in the upper 100 to 150 feet of sediment in the Gulf of Mexico were known, it should be possible to estimate the strength of sediments for preliminary design planning. It is almost certain that the rate of change of geologic characteristics of sediments from place to place in the shallow water areas of the Gulf is slow and that once criteria for correlating geologic characteristics of sediment with strength are available, it should be possible for the geologist effectively to help the engineer in planning offshore drilling operations.

REFERENCES

Casagrande, A. (1936). The determination of the preconsolidation load and its practical significance: First International Conference on Soil Mechanics and Foundation Engineering, vol. III.

First and Second International Conferences on Soil Mechanics and Foundation Engineering (1936). Proceedings: Harvard University (3 vols.) and Rotterdam (1948) (5 vols.) Symposiums on fundamentals of soil mechanics.

Kelley, W. P. (1948). Cation exchange in soils: Amer. Chem. Soc. Monograph 109.

Krumbein, W. C. and Monk, G. D. (1942), Permeability as a function of the size parameters of unconsolidated sand: Amer. Inst. Min. Engr., Trans., vol. 152, pp. 153-163.

Krumbein, W. C. and Pettijohn, F. J. (1938). Manual of sedimentary petrography: Appleton Century Co., pp. 228-267.

Russell, R. J. (1952). Recent geology of coastal Louisiana: Second Conference on Coastal Engineering, Council on Wave Research, Berkeley, California.

Terzaghi, K., and Peck, R. B. (1948). Soil mechanics in engineering practice: John Wiley and Sons, pp. 1-136.

Trask, Parker D., and Rolston, J. W. (1950). Relation of shear strength of sediments to water content and grain size: Science, vol. 111, pp. 666, 667.

Trask, Parker D., and Rolston, J. W. (1951). Engineering geology of
San Francisco Bay: Geol. Soc. Amer. Bull., vol. 62, pp. 1079-1109.

Trask, Parker D., Phleger, F.B., and Stetson, H. C., (1947). Recent
changes in sedimentation in the Gulf of Mexico: Science, vol. 106,
pp. 460-461.

Umbgrove, J.H.F. (1930). The amount of maximum lowering of sea level
in the Pleistocene: 4th Pacific Science Congress, Proc., vol. 2,
pp. 105-11o.

Chapter 13

THE FOUNDATION PROBLEMS ON THE GULF COAST*

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The foundation problems of the coastal region of the Gulf of Mexico are unique. Normally, a coastal region is thought of as the land area, such as a plain, adjacent to a body of water. Such a region usually is somewhat regular in its geology and because of the natural resources, terrain or climate may be given to a relatively common industry involving a somewhat similar development throughout. The coastal region of the Gulf of Mexico, as regards the United States, violates this criterion in a multitude of ways. The region is not limited to the coastal plain bordering the Gulf of Mexico, by any means, but, rather has been broadened by our commerce and the need for the development of natural resources to also embrace, the delta areas and offshore belt extending to the limit of the continental shelf, lying as far as 70 miles from the shore. The delta areas have long been avoided in the past by industry of all types; that is, with the exception of the fishing industry, because of the unstable nature of the foundation media. Likewise the continental shelf area normally is not considered for industrial development because of the availability of the more desirable coastal plain. However, the quest for natural resources, like sulphur and petroleum, in spite of the efforts toward Federal Control, has made necessary the solution of very extraordinary foundation problems in this offshore area. In addition to the foregoing unusual aspects of the foundation problems of the Gulf Coast, the coastal plain is unusual in itself because this region at one time formed the floor of the Gulf of Mexico and, as the sea receded or the land was uplifted, the residual sedimentary soils have been drained and desiccated to result in unusual formations that serve as foundation media for the industrial and domestic developments of the region. The foregoing factors combine to make the foundation problems of the Gulf of Mexico Coastal Region very interesting.

Consideration of the foundation problems of a region must logically start with a brief review of the geology of the region. The geology of the Gulf Coastal Region is fascinating. It may be divided into three broad divisions from a foundation point of view. The first broad division embraces the coastal plain area that lies some miles inland at the present time. The surface and near surface strata dip toward the Gulf. These strata are usually alternate layers of sand and clay which are well drained and somewhat desiccated to a relatively advanced state of

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consolidation, so they thus serve as reasonably desirable foundation media. The second broad division embraces the area of the former deltaic deposits of the rivers emptying into the Gulf, which lie between the coastal plain and the present shore line. This area exists in the form of a belt bordering the Gulf, varying width from a few to about 70 miles, with the exception of the major contributor; namely the former delta of the Mississippi River which extends inland many miles. This area is very significant because it is the one in which great development of natural resources has occurred and also has been the area in which the industrial development has been and will continue to be very great. The materials comprising the former deltaic deposits are generally sufficiently matured by drainage and desiccation to now be in a reasonable stable condition; however, somewhat less stable as foundation media than the materials comprising the coastal plain. The third broad division embraces the presently growing deltaic areas forming at the outlets of the rivers discharging into the Gulf of Mexico. These areas are of two categories; namely, those that have grown above Gulf level and those that have not. Those that have grown above the level of the Gulf have only begun to mature; consequently, they are in the process of draining and consolidating and, at present, are in a relatively unstable condition as regards foundation media. Those that have not grown above the level of the Gulf are still growing and have not started to mature. They have not begun to drain except for that resulting from consolidation caused by the weight of the material itself. These materials are in a very unstable state to considerable depths requiring very careful analysis when used as foundation media. The major element of this area is the offshore deltaic deposit of the Mississippi River that is fan-shaped and slopes gently toward its fringe, which lies as much as 70 miles offshore. Extensive development of the natural resources such as petroleum and sulphur have been initiated in this area.

The development of the Gulf Coastal Region for many years was paced by the growth of the agricultural industry and the commerce that flowed both to and from it and the central and southwestern portions of the United States. The discovery of very extensive natural resources, of which low cost natural fuel is an important one, has caused an industrial invasion of the region. Many well established industries have moved into the region and many new industries have been created because of the fuel and other natural resources discovered. Accompanying the industrial invasion and development of the resources there has been a great growth of the foundation problems.

The foundation problems have varied with the advance of the industrial development, the types of structures required and with the geology of the region in which the industry is located. The initial development of the petroleum, gas and sulphur industries occurred in the former or old deltaic area of the region. Likewise the industrial development had its beginning in this same area where it will be continued because it is the logical location as regards transportation, centers of

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population and central location with respect to the natural resources. However, with the ever increasing demand for natural resources, there has been an expansion of the petroleum, gas and sulphur industries, particularly as regards the production faoilities of these resources, toward the Gulf of Mexico into the recent deltaic area and, further into the now forming deltaic areas; that is, into the waters of the Gulf of Mexico with some of the structures located as much as 30 miles from shore.

The Civil Engineering profession has followed the old economic law of "Supply and Demand", but in reverse. For with the creation of a demand, the profession has supplied the technical and economic solu- tions for the problems. Accordingly, during the late 1920's when the industrial and natural resource development was beginning the foundation problems were modest and our knowledge was limited, solutions that were thought to be simple and economic were used. For example, it was the rather common practice to use piles for the substructures of the then larger structures and shallow spread footings for the smaller structures involved. However, as the demand increased for larger and more econom- ical structures in the old deltaic area, knowledge of the foundation media and means of economically transferring structural loads at these media had likewise increased so that improved methods were created to replace the expensive and questionable pile substructures except, when especially suited for a specific purpose.

The improved methods of design and construction are the outgrowth of improvements in the methods of exploration, sampling and testing so that knowledge of the pertinent factors of the foundation media can be gained as required for rational analysis. Further, knowledge has been gained of the intensity and distribution of the stresses created in soil masses by the various types of substructural elements by the re- searches of Dr. Leo Jurgenson and the Waterways Experiment Station so that rational analysis is possible now for foundation problems. Con- current with the accumulation of this combined knowledge there has been the development of the machines for boring shafts, with enlarged bell- shaped bottoms, both large (maximum of 12 ft. diameter) and small to shallow or large depths (maximum of about 80 ft.) to transfer the struc- tural loads through the unstable surface strata to the underlying con- solidated and stronger strata capable of supporting the loads with the factor of safety desired. For circumstances requiring footings larger in area than can be produced by the machines, open excavations for the conventional spread footings are normally used.

The important problems to be solved in the design of this type of substructure is the determination of the depth to which the footings should extend and their dimensions. Usually one would think both of these factors would depend entirely upon the strength of the materials forming the foundation media. However, in the old deltaic area many of the surface clays, such as the Houston or Beaumont Groups, experience appreciable volume changes accompanying seasonal variations of moisture content. Accordingly it is important to extend the substructural

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elements into the zone of the capillary fringe of soil moisture so as
to preclude the effect of volumetric changes. In addition it is impor-
tant to either provide a suspended floor system or a floor that is in-
dependent of the structural frame for the ground or basement floors.
Further the grade beams for the structural frame should always be de-
signed so as to be protected from the swelling clays, which have been
observed to exert vertical pressures of the order of ±10 tons per sq.
ft. It has been interesting to observe the scars left in the clay
formations experiencing volumetric changes. These clays are locally
termed "slickensided" because in shrinking many shear fracture surfaces
are formed throughout their mass. Accordingly shear strength, as de-
termined by the unconfined shear test, are low and very erratic, how-
ever the identical soil within the zone of the capillary fringe usually
is relatively homogenous and possess considerable shear strength. A
plot of the unconfined shear strength with respect to depth usually
serves as an excellent means for determining the depth to which the
"slickensided" condition extends. Subsequent to the defining of the
depth of the fractured zone, the selection of the stratum in which to
found the footing is a matter of economics. The determination of the
diameter or area of the individual footings can be made in accordance
with the procedure described in "Soil Mechanics in Engineering Practice"
by Terzaghi and Peck.

The foregoing procedure has been used by the author for the solution
of foundation problems for industrial and office buildings, large surface
storage tanks, heavy machinery foundations, large incinerator plants and
a multitude of other structures. The Texas Highway Department has made
extensive use of essentially the same procedure for the substructures to
support the bridges and elevated highways for the urban expressways of
Houston and other cities in the state.

The state of consolidation of the foundation media in the old delta-
ic formations is generally so well advanced that the estimation of the
settlement of structures in this area is of secondary importance; that
is, unless such is required for a special purpose. The procedure usually
used conforms to any of those described in well recognized texts on soil
mechanics. A careful check of this matter is made for important struc-
tures to determine the influence on the structure. Numerous reports of
observations of structures in the area of interest have shown the estim-
ates of settlement based on the Terzaghi Theory of Consolidation, are
valid.

The foundation problems experienced in connection with port struc-
tures, such as wharves, are significant to the development of regions
like the Coast of the Gulf of Mexico. You may recall from your memory
of the maps of this coast that most of the harbors are located on rivers
or bays. At these locations the sites for the port structures are fre-
quently either in the old deltaic formations or near their junction with

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the recent formations of this nature; consequently it would be opportune to discuss this type of problem.

The majority of the wharves constructed in the coastal region during the 1920's and 30's were the conventional bridge type formed of treated timbers and supported by wooden or concrete piling. At the time of their design and construction, knowledge of foundation engineering was rather limited when considered in the light of the present day status, so it was natural that conventional structures would be used. However to meet the present day criterion of "economy through engineering", it has been necessary to mobilize and apply all of our knowledge to establish the best suited structure for this purpose.

An excellent example of what can be done is the new Wharf 16 recently completed for the Port Commission of Houston. It is the second of two such structures that are new to the Gulf Coastal Region differing radically in principle from their predecessors. The prime purpose in initially advocating the new approach to this problem was based on foundation difficulties previously experienced nearby which resulted in serious damage to former wharves of the conventional type, thus a "red" traffic light was glowing in our faces when the problem was first approached. A well planned and executed examination of the site immediately adjacent to the site of the former structures disclosed the presence of a stratum of fine-grained sand, about four feet thick existed between elevations -26 ft. to -30 ft. At the time the original structures were constructed the harbor floor was at elevation -25 ft. Subsequent dredging to permit navigation by larger vessels extended the harbor floor to elevations -30 ft. and later at -36 ft. elevation. After the latter dredging and, it is understood, accompanying the difficulty with the original structures, soundings are reported to have disclosed the formation of a sizeable deposit of the previously mentioned sand over the floor of the harbor. Observation of the surface features of the sites of the original structures disclosed the presence of numerous and extensive earth slides whereby the graded areas had moved downward and harborward. These facts showed that, in the selection of the substructure for this new wharf, consideration must be given to the extension of the substructure through any sand strata that might be subsequently exposed by later improvements to the harbor, or, provide a means for the complete confinement of such sands that might be displaced by piping.

The ultimate design recommended and used was a cellular bulkhead formed of steel sheet piling or, in other words a coffer-dam type of substructure. The sheet piling forming each cell were planned to extend through the fine sand strata into the structurally sound underlying clays to form a double barrier to piping of sands into the harbor. The cellular bulkhead was greatly reinforced and appreciable economy thereby affected by filling the cells with a selected sand that was placed in layers and densified in place by vibratory methods. The use of this type of cell filling in a densified condition resulted in an increase of the strength and stability of the structure of approximately 20 per cent. The void between the bulkhead and a natural bank was also filled with densified

sand placed in a like manner. The topping of the sand backfill and over the cellular bulkhead was formed of compacted earth to form the subgrade for the pavement of the open apron of the wharf.

The economy of this new type of wharf or port structure over the conventional bridge type may be of interest. Competitive construction bids were secured for the new and the old types. A net saving of approximately $550,000 was achieved by the use of the cellular bulkhead type.

The editorial comment on the type of wharf just described, appearing in the English magazine "The Dook and Harbour Authority" published in London, August 1951 is of interest. It is quoted as follows: "It seems that cellular bulkhead substructures for wharves have possibilities in certain situations, which might well be further explored in this country."

The limitation of time precludes treatment of many of the interesting foundation problems arising in the old and new deltaic formations of the Gulf of Mexico Coastal Region. To briefly enumerate some of the types of structures involved may suffice. They are the stability examinations for the design of the banks for channel improvements and new waterways; fresh water reservoirs that are being required more and more frequently by industry in this region; large petroleum products and chemical types of industrial plants; and highway embankments and bridge structures.

One of the new types of foundation problems arising in the offshore and presently forming underwater deltaic area is most intriguing. There are others, I assure you; however, the drilling platforms required for offshore production (oil and gas) has been discussed extensively in current engineering literature and cannot be passed in this paper without brief reviews. The quest for more natural resources has prodded the civil engineer to seek solutions to this foundation problem and much has been learned from these studies; however, much remains to be learned.

The majority of the sites for these structures are in the unconsolidated deltaic deposits, which are still growing, and form the bottom of the Gulf of Mexico. Accordingly, the materials comprising the foundation media have not been drained or consolidated except under their own weight and that of the overlying materials whose effective pressure is very low because of their submergence. Therefore the foundation media possess relatively low strength.

The loads to be supported by the substructures are large, approaching the magnitude of those experienced with sizeable bridges. These loads are caused by the heavy drilling machinery required for the drilling operations and for the production operations if the wells develop as their planners hope.

The pile substructure has been used for these structures. It lends itself to this use admirably because, if properly designed, it is capable

of transferring large loads through weak materials to stronger and deeper portions of a foundation medium. Further, the pile is capable of distributing its applied load to the material penetrated either through friction or by point bearing in the event strong materials are encountered.

The design of the penetration depth of piles as for specific vertical static load supporting capacity in the past has largely been based on arbitrary formulae evaluating forces created in driving a pile and the penetration achieved. Many formulae have been proposed, used and their results evaluated with somewhat disappointing results if the information was evaluated carefully. It is understood these arbitrary formulae were used in some instances for the early platforms designed and constructed. In other instances the piles were driven to very great depths to essentially refusal in order to be sure of their supporting capacity. However, the matter of economics began to have its influence and as a consequence the spotlight of engineering attention has been directed on this problem with gratifying results. It may be interesting to note at this point that the investment involved in an offshore drilling platform alone is sizeable for it may vary from a quarter to a million and one-half dollars.

It was my pleasure about a year ago to study the substructural aspects of two pile platforms, one constructed and the other proposed, in the new deltaic area of the Mississippi River. Complete information on the kind, arrangement and strengths of the foundation media were available. The basis for the approach to the problems was that the vertical supporting capacity of a pile, whether it be formed of steel, concrete or wood, depended in these instances upon the available soil friction about the periphery and for the length of the penetration. The friction developed would be influenced by the material forming the surface of the pile. For example, in the case of a concrete pile if the skin surface was very smooth, such as would result from vibrated concrete placed in a metal form, driving would be facilitated but to produce load supporting capacity long penetration would be necessary because of the low soil friction. On the other hand, if the skin of the pile was roughened by placing burlap against the smooth metal forms when the concrete was placed, so as to produce a rough textured surface, the soil friction would be mobilized to a larger extent. In the case of steel piling the normal oxidation of the surface of the steel serves to provide a bond with the soil penetrated so as to mobilize the shear between itself and the soil. Further, the penetration of a pile into soil produces a rapid consolidation of cohesive soils adjacent to the pile and to densify cohesionless soils due to the volumetric displacement, both of which increase the shear strength of the soils immediately adjacent to the pile.

The results of both analyses showed that rational vertical loads supporting capacities could be developed. Subsequently, vertical loading tests of similar piles used in the same general vicinity showed

that capacities determined by the foregoing procedure were within 10 to 15 per cent of the analytical results and for vertical pulling tests, the results were within 7.5 to 10 per cent of the analytical results. Confirmation of analysis of the order indicated is very gratifying.

The overall security of the pile substructures for the offshore drilling platforms is not only a function of their vertical stability but also the lateral stability of the piles. The structures located in the open reaches of the Gulf of Mexico are exposed to storm waves of hurricane intensity and should be designed to satisfactorily meet these conditions. Unfortunately at present, technical knowledge is very meager about the horizontal forces the structures are actually required to withstand. The assumption made by designers regarding forces exerted on the structures by the hurricane winds and waves vary with the designer. Further, the assumptions made by the designers regarding the deflections, points of fixity and how the horizontal forces are transmitted to the foundation media by the pile substructures vary somewhat in the same manner as those pertaining to the forces themselves. This lack of knowledge has been appreciated by many interested in the problem and numerous efforts have been made to initiate research that would produce the information desired. These efforts are beginning to pay dividends, for research on a modest scale is being conducted by several interests. It is hoped that by the time the ownership of the tideland area has been established the sorely needed information will be available to the profession for intelligent design purposes.

The foundation problems in the new deltaic area have been treated tersely in this paper with no intention to slight them; however, the space available precludes extensive treatment. There are many other interesting problems that warrant mention at least; a few of them are: the design of economical foundations for underwater pipelines to serve the offshore oil and gas wells; and the design of the substructures for the pumping, and processing storage facilities for oil and possible sulphur in this region.

It is hoped the thoughts expressed in this paper will stimulate the thoughts of others interested in the problems of this nature so that the progress made to date may be greatly extended in the future in achieving economical designs for the problems encountered in the practice of civil engineering.

Chapter 14

EROSION ALONG THE ILLINOIS SHORE OF LAKE MICHIGAN

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In the consideration of problems of coastal engineering in general, and of shore and beach erosion in particular, one is quite naturally apt to immediately focus attention upon our ocean shores to the exclusion of inland areas.

Many are perhaps not appreciative of the fact that the five inland fresh-water lakes comprising the Great Lakes system are bordered by eight states having a combined length of shore line of approximately 3,000 miles. (See Fig. 1) Only in recent years has there been a general awakening of interest in the many and varied problems of erosion which occur along these inland coasts.

The purpose of this paper is to summarize the problems existing on the Illinois shore and to outline the steps which have been taken at the State level in seeking a solution to those problems.

DESCRIPTION OF SHORE LINE

The Lake Michigan shore line of Illinois, while only 53 miles in length, has been highly developed into industrial, residential, and recreational areas. Located thereon are the cities of Chicago, Evanston and Waukegan and many lesser communities, all having an approximate aggregate total 1950 population of 3,800,000 and having property subject to the effects of erosion valued at in excess of $125,000,000.

In prehistoric times, a glacier covered the Great Lakes area, extending as far south as southern Illinois. The present-day Lake Michigan was dug by the gouging action of tongues of the glacier. The various tongues alternately advanced and retreated, depositing at their termini ridges known as moraines. The final glacial tongue which entered Illinois deposited the Lake Border Morainic system, which follows in general the shape of Southern Lake Michigan, as shown in Figure 2.

It was this glacial action and the associated stages of the glacial lake which give rise to the present composition of the shores of Lake Michigan.
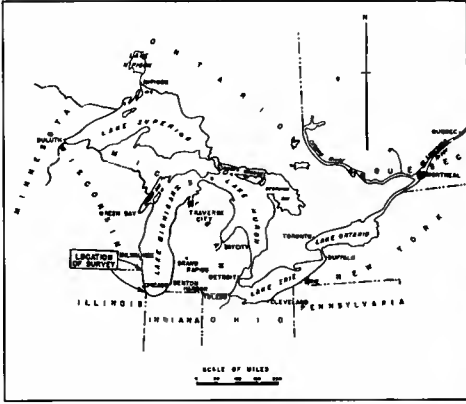
Fig. 1
Location Map
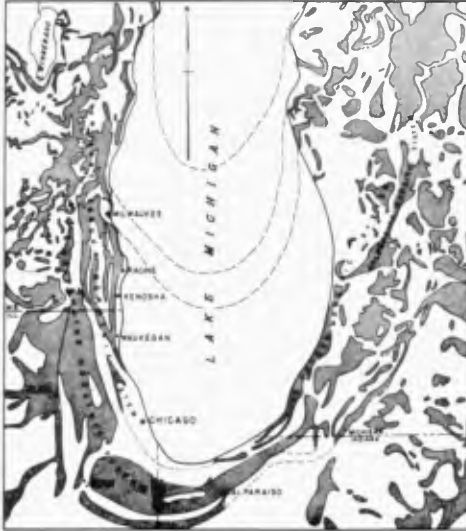
Fig. 2
Lobate distribution of glacial
moraines around the southern
part of Lake Michigan. (After
Leverett, Alden, and Weidman,
U. S. Geological Survey Prof.
Paper 106, Pl. XXIII, 1918).
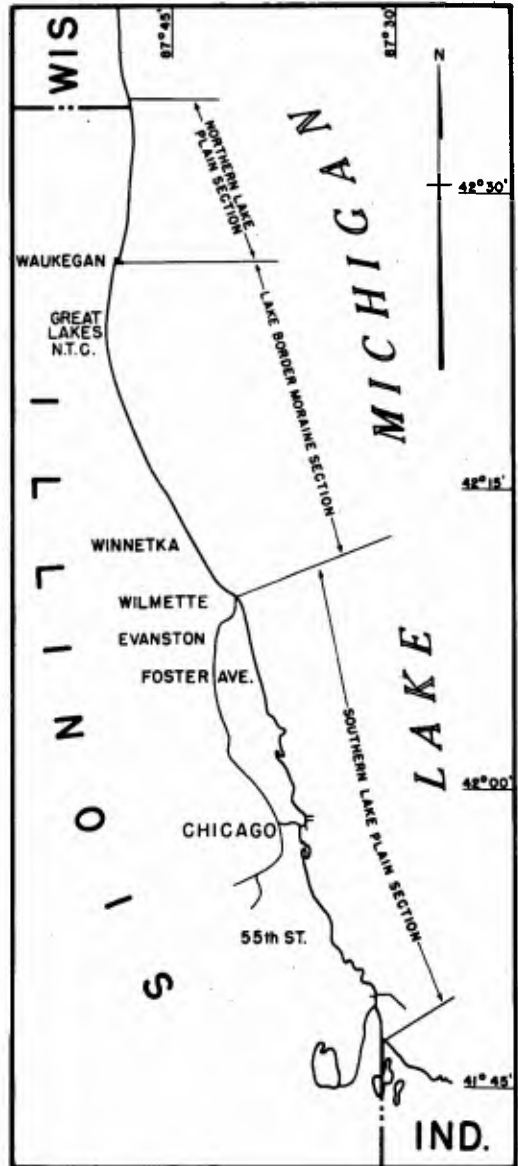
Fig. 3
Physiographic Divisions – Illinois
shore of Lake Michigan.

COASTAL ENGINEERING

Based upon a combination of geomorphic and other features, the Illinois shore has been divided into three major segments, as follows: (See Fig. 3).

1. The Northern Lake Plain section extends from the Illinois-Wisconsin line to Waukegan. In this reach, the shore is fronted by a glacial lake plain generally on the order of 5 to 15 feet above the present lake, marked locally by ancient beach ridges and dunes, and composed of deposits associated with the earlier glacial lake or its shores. This section is generally eroding except immediately to the north of the Waukegan Harbor structures, which form an impounding area for the littoral drift. The present erosion essentially represents a recapture of the glacial lake deposits by the present lake.

2. Continuing southward, the reach from Waukegan to Wilmette, designated as the Lake Border Moraine section, is featured by the Lake Border glacial moraine which intersects the shore, giving rise to bluffs up to 90 feet in height composed of glacial till and associated outwash deposits. This section is generally eroding slightly as far south as southern Winnetka; from that point to Wilmette Harbor, there is an impounding area largely created by the harbor structures.

3. In the Southern Lake Plain section, extending from Wilmette southward to the Illinois-Indiana line, the shore is composed of the old lake plain of the glacial lakes, with deposits of ancient beaches, dunes, and lacustrine clay.

Under natural conditions, the reach from Wilmette to 55th Street, Chicago was largely subject to marked erosion, while that from 55th Street to Indiana was a natural impounding area for the littoral drift from the north. At the present time, the shore from Wilmette to Foster Avenue, Chicago has largely been protected by groins and bulkheads. Southward from Foster Avenue, the shore is largely one of artificial fill resulting from operations of park districts and industry.

EROSION PROBLEMS

At the present time, the Illinois shore northward of Foster Avenue, Chicago, is generally in a state of erosion save at the up-drift side of the harbor structures at Waukegan, Great Lakes Naval Training Center, and Wilmette, and at the projecting bulkheaded fill at Foster Avenue.

EROSION ALONG THE ILLINOIS SHORE OF LAKE MICHIGAN

The littoral drift, which is predominantly from north to south, is, at best, relatively lean as compared to the ocean shores. The available supply of drift is derived almost exclusively from shore erosion, since there are no streams of consequence entering the lake along this reach of the shore. A large portion of the drift is impounded by the previously mentioned harbor structures, thus impoverishing the shores southward thereof. In addition, the configuration of the shore line at Evanston is such that there is an apparent tendency for the available drift moving to that point to be deflected lakeward into deep water, thus starving the beaches to the south.

Erosional processes also vary to a very considerable extent with lake stage. During periods of high lake stage, the bluffs are within closer reach of wave attack, with a consequent increase in the rate of erosion. During periods of low stages, a wider beach is exposed and hence the bluffs are better protected.

Another problem for the State arises from the exercise of the permit powers conferred by statute. The proper analysis of permits for construction of groins, jetties, and other structures is greatly hampered by a lack of knowledge of the probable effectiveness of the proposed structure for its intended use and of its probable effect upon the property of others.

Time does not permit a detailed discussion of the various points and types of damage. Suffice it to say here, that erosion has been, and is, responsible for damage to recreational beaches, water plants, sewer outfalls, protective structures which have been constructed in the past, and the property and homes of many who thought that they were building at a safe distance back from the shore.

BEACH EROSION INVESTIGATIONS

COOPERATIVE STUDIES

It was not until the early nineteen-forties that serious attention was given to these problems on the State level. Preliminary consideration of the problem made it apparent that nothing less than a complete and far-reaching investigation could provide a basis for solution and that the desired results could perhaps best be attained through a cooperative beach study with the Federal Government.

Accordingly, the 64th General Assembly appropriated the sum of

$62,500 in 1945 for the purposes of a cooperative investigation. A formal agreement between the Beach Erosion Board, Corps of Engineers, Department of the Army, and the State of Illinois, acting through its Department of Public Works and Buildings, Division of Waterways, was executed in February 1946. The agreement specified that the purpose of the study was to determine the best method of preventing further beach erosion, of stabilizing existing beaches, and of restoring eroded and damaged beaches.

In addition, a supplemental agreement was signed in June 1947 whereby the scope of the study was enlarged to include the development of a plan of improvement and the determination of the extent to which the Federal Government could participate in the cost of improvement under the provisions of Public Law 727, 79th Congress.

The final report on the study has just recently been forwarded to the Secretary of the Army by the Chief of Engineers. The report sets forth in detail the results of the various studies involved and recommends Federal participation in the construction of shore protection measures for public property at Lake Bluff, Lake Forest, Winnetka, Kenilworth, Evanston, and Chicago.

Recommended protection measures at these locations range from groins, jetties, piers, and bulkheads to the placement of sand fill. In addition, the report suggests protective measures for adoption by owners of private property consisting chiefly of riprap or short groins.

INVESTIGATIONS BY STATE OF ILLINOIS

Subsequent to the initiation of the cooperative beach erosion study, it was found that reliable data on wave height and directions and the source, character, and movement of beach material were entirely lacking and that the most recent hydrography was dated 1909-11. Limitations of time, funds, and scope of project made it necessary that the cooperative study and report proceed on the basis of available information, supplemented by data obtained during the course of the survey.

Recognizing, however, the need for more information as to the fundamental processes involved in erosion problems, the 66th General Assembly appropriated the sum of $35,000 in 1949 to carry out basic studies relative to shore erosion processes, either independently or in cooperation with the Federal Government and the 67th General Assembly appropriated a like sum this year for the same purpose.

Contact with the Beach Erosion Board with reference to an additional cooperative study developed the fact that the Board lacked the necessary funds and it was then agreed to enter into an informal agreement whereby the Beach Erosion Board would act as consultants in developing and executing a program and that the State, through the Division of Waterways, would undertake the execution of all field and office investigations.

The program was initiated in early 1950 and has two major objectives, as follows: (1) The study of the character, source, and movement of beach material and (2) the determination of the magnitude and direction of the waves impinging on the beach.

The undertaking of such a program on the part of the State proved to be quite a task in that studies of this type still may be said to be in the pioneer stage even on the sea coasts and doubly so on the Great Lakes. The State lacked both the necessary equipment and the personnel trained in this type of work.

The equipment problem has been largely overcome by the acquisition of the necessary items, as will be discussed later, and the problem of personnel is being met by the development of our present staff through study, indoctrination courses, and experience.

STUDIES ON BEACH MATERIALS

Experience to date in prosecuting a study of the first objective, namely, the character, source, and movement of beach material, has been that such a study presents so many facets and ramifications that it is extremely hard to choose a course to pursue and, having done so, to keep on that course.

Field work on this phase of the program in 1950 consisted of taking soundings and bottom samples to a depth of 60 feet on ranges located approximately one mile apart with a DUKW and fathometer leased from the Corps of Engineers and using a drag-type sampler recommended by the Beach Erosion Board. Profiles were also run on the shore end of the ranges and beach and bluff samples taken. In addition, bluff samples were taken along the Wisconsin shore as far north as Milwaukee at intervals of one mile or less, since this reach is the apparent source of much of the beach material.

Laboratory work on the 1950 samples was confined, during that

year, to grain-size analysis, using the 2 series sieves down to the No. 16 and an Emory settling tube for the remainder.

A comparison of the range profiles for the cooperative study in 1946 and the State's survey of 1950, together with the results of visual observation in the spring of this year lead to the conclusion that there is apparently little movement of material lakeward of the 5-fathom line and also that profiles at one-mile intervals on an annual basis were not adequate to determine the movement of material.

Therefore the sounding and sampling program for this year was revised to include a considerable number of intermediate ranges together with the taking of profiles on groups of seven ranges spaced at 200-foot intervals along the shore in critical locations.

In this connection, it was necessary for the State to procure equipment for the 1951 work and there has now been acquired an Army DUKW (amphibious truck), a fathometer, and two-way radio equipment consisting of a mobile 30-watt set mounted on the DUKW, and three portable sets for use on shore to maintain the DUKW on course, as shown in Fig. 4.

The acquisition of this equipment delayed the 1951 field program until September 15 and hence the work was not completed in its entirety because of the advent of bad weather. The work did, however, cover the northern end of the shore line where the most serious erosion is taking place.

Early in the spring of 1951, a conference was had with Dr. Martin A. Mason, then with the Beach Erosion Board, and at his suggestion, the program of sample analysis was considerably expanded.

Samples are now being analyzed as follows:

a. Sieve analysis, using the 2 series sieves throughout the range down through the No. 325 mesh.

b. Separation of light and heavy minerals, using bromoform.

c. Magnetic mineral separation, using a weak magnet.

d. Carbonate separation, using diluted hydrochloric acid.
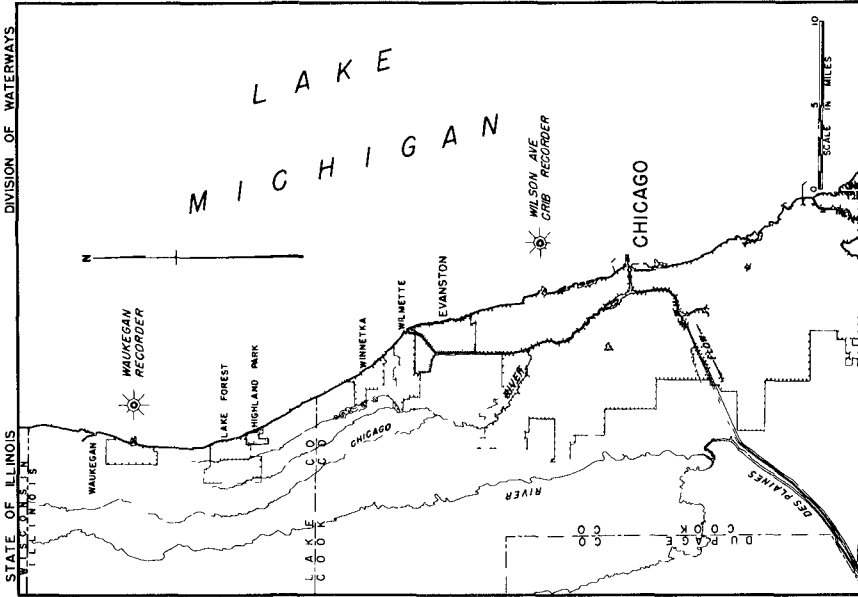
Fig. 5

Location of Lake Michigan wave recorders.
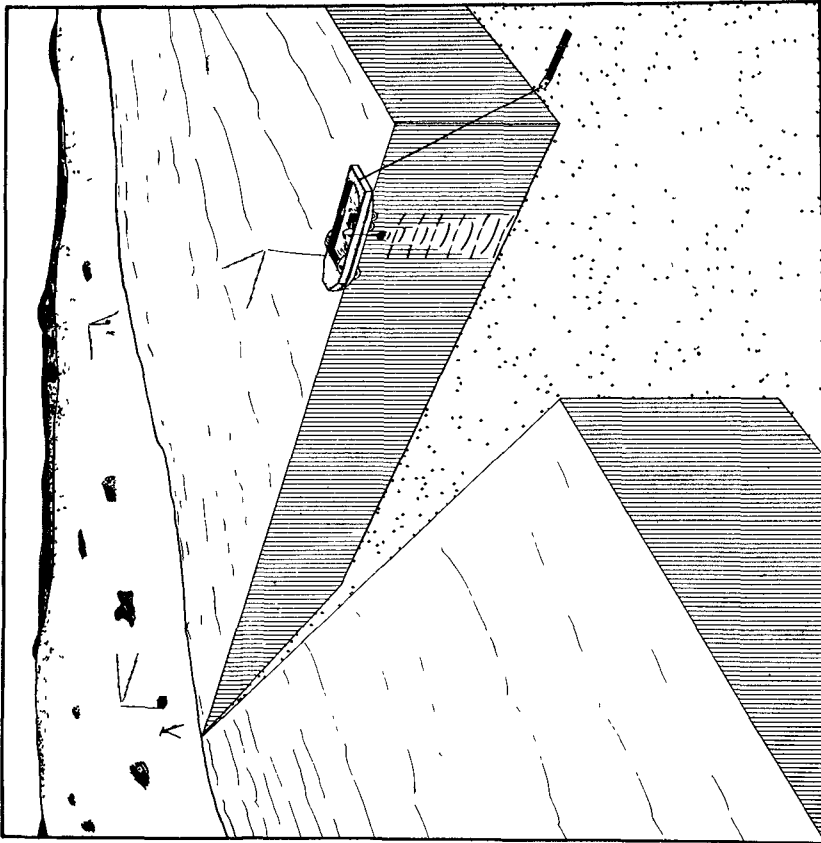
Fig. 4

Diagram of sounding and sampling operations from amphibious truck.

This work is also being done by the staff, working in a small laboratory set up for that purpose. The analysis of these samples is as yet far from complete and hence conclusions cannot yet be drawn.

In planning for future work along these lines, it has been concluded that even the program of sounding and sampling carried out this year is not perhaps the best for the purpose. It is now believed that since the State has acquired it's own equipment and can undertake operations at its discretion, future operations of this nature will be based on the study of both onshore and offshore changes taking place at selected locations during storm periods rather than simply on an annual basis.

STUDIES OF WAVES

In attacking the second objective of the present studies, that is, the determination of the magnitude and direction of the waves impinging on the beach, considerable progress has been made.

The Institute of Engineering Research of the University of California has developed and delivered to the State, one Mark IX Shore Wave Recorder system, which was installed in Lake Michigan in June 1951, about 300 feet northeast of the Wilson Avenue Waterworks Intake Crib of the City of Chicago, approximately 2 miles offshore and in 35 feet of water. In so far as is known, this is the first wave recorder to be installed in the Great Lakes. The University is also under contract for a similar unit which is to be installed off Waukegan Harbor next spring. The relative location of these installations is shown in Fig. 5.

The Wilson Avenue Crib unit is presently operated at a chart speed of three inches per hour with two daily periods of 15 minutes each at a speed of three inches per minute at noon and midnight.

It is planned that both the Wilson Avenue Crib and the Waukegan Harbor station shall both be more or less complete in so far as wind, waves, and lake stages are concerned. To that end, both stations have been equipped with wind recorders, which provide a continuous record of both wind direction and velocity, and with long-term stage recorders which provide a continuous record of lake stage.

While the Wilson Avenue recorder has been in operation for only a few months, the records obtained to date indicate that data as to the generation and duration of storm waves with respect to wind may be of particular interest in that the entire picture of storm occurrence may be studied from one period of flat calm to another.

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It is unfortunate that the wave recorders cannot be operated during the winter months. The hazard to the underwater pressure heads from floating ice is such that they must be removed for the period November-April of each year. There is one compensatory feature, however, in that during those months the beaches are generally heavily covered with ice and hence suffer little damage from wave action.

Visual observations of wave direction are being made by means of a transit equipped with a sighting bar from a station located on the bluffs at Lake Forest and it is anticipated that another station will be selected in the near future.

In the course of the previously mentioned conference with Dr. Mason early this year, it was concluded that the preparation of wave refraction diagrams for the entire Illinois shore would be of value in charting the program of field investigations, even though specific information on wave height, period, and direction was yet lacking.

The lack of personnel qualified to prepare such diagrams made it necessary that one of the engineering staff attend the beach erosion school held in Washington, D. C. in March of this year for training in those procedures. Dr. Mason, and the Beach Erosion Board were most co-operative in making the necessary arrangements for this training.

Following this training, the staff prepared wave refraction diagrams for various wave periods for the entire Illinois shore line. The so-called crestless or orthogonal method was used throughout.

In the course of this work, it was found that the complex pattern of sink holes and small offshore lumps made it necessary that much idealization of the submarine topography be done. In addition, the latest available hydrography was dated 1909-11 and had to be drastically revised to conform to the profile soundings of 1950.

These diagrams were, of course, entirely preliminary. The lack of definite information regarding average wave periods and predominant direction of travel, coupled with the antequated hydrography, necessarily renders such diagrams of doubtful value, but they have been useful in the charting of the field program. However, data from the wave recorders, together with the results of pending new offshore hydrography by the U. S. Lake Survey, coupled with the present soundings, will permit the preparation of more complete and accurate diagrams in the future.

CONCLUSION

It is hoped that this resume of the activities of Illinois in the study of erosion problems on Lake Michigan may have been of some interest in pointing out the steps which may be taken at the State level. The present study is as yet, not sufficiently advanced to predict the results that may be attained. Much remains to be done in what is, for Illinois, a new field.

However, it is felt that the results of the study will be of great value in charting the course of the State's activities in preserving and augmenting the important natural resources inherent in its shores, not only in planning and participating in such remedial and protective measures as the General Assembly may deem advisable but in the regulation of the location and type of private construction through the permit powers granted by statute.

Chapter 15

SAND BY-PASSING PLANT AT SALINA CRUZ, MEXICO*

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Mexico D. F., Mexico

The Port of Salina Cruz, in the State of Oaxaca, Mexico is en-
tirely artificial. It consists of an outer harbor and a basin divided
by two large wharves on which stand six double warehouses. For a time,
jurisdiction on this port was equally divided between the Administration
of Free Ports and the Federal Fiscal Authorities, but at present, by
Presidential decree, the entire port is under our Free Ports Adminis-
tration. On the west end of the outer harbor, a dry dock able to handle
boats up to 18,000 tons has been operating since it was built. On the
same side of the outer harbor, terminal installations of Petroleos
Mexicanos handle all shipments of crude oil and gasoline for the Pacific
Coast of the country.

The English concern that originally planned the port works of
Salina Cruz, had in mind only the breakwater starting from Beacon Hill
and extending sufficiently into the sea to protect the entire length
of the wharves against the seas and winds from the south, thus forming
an outer harbor with its entrance on the east. However, when construc-
tion of the breakwater had progressed to the point where it now turns
southward, it was observed that, during stormy weather and heavy seas
from the south, the space between this section of the breakwater and
the wharves, or what was then becoming an outer harbor, would fill up
with sand. The English engineers grasped the enormous importance of
the sand drift from the west. Upon realizing this, they changed the
direction of the breakwater towards the open sea and were forced to
construct another breakwater on the eastern side, thus forming the
present outer harbor. Sand began to deposit from Beach Hill to the
extreme point of the breakwater forming a huge sand fill, sometimes
filling to depths up to 15 fathoms. Once this deposit started to accu-
mulate to overflowing, sand would invade the outer harbor making it
necessary to keep a dredge working continuously, as it has in the past
40 years, in order to pump it towards the open sea. This work has cost
the Mexican Government over 50 million pesos with the bay constantly
running a risk, as the sand, accumulating in the outer harbor, closes
up the channel and makes navigation difficult and dangerous through
tortuous channels from the entrance to the basin proper.

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*(Editor's Note: This installation is considered to be one of the
first large scale stationary dredges in the world. The effectiveness
of this plant in accomplishing its purpose is not known at this time;
however, its performance is being closely observed by the Beach Erosion
Board, and it is suggested that future publications of that agency be
consulted for an evaluation of the effectiveness of this unique
installation.)

During the last few years dredging has been sporadic, a fact which has, unfortunately, smirched the reputation of the port since large boats cannot dock there, thus paralyzing the commercial development of the entire region. It is well known that Salina Cruz and Puerto Mexico, each on the terminals of the Isthmus Railroad, are geographically strategic points in world communications and are very important throughways to increase international trade and specially United States intercoastal trade.

The sand drift from western seas that kept clogging the bay of Salina Cruz stood as a great challenge to Mexican engineering. It has been the good luck of the Administrative Board of Free Ports to be able to attack the problem in search of a solution even before the Ministry of Marine was created.

The work has progressed with full approval of various Presidents of the Republic who have shown interest in our plans, specially President Miguel Aleman who has given full support to the program. We are also greatly indebted to senor Beteta, Minister of Finance, whose staunch backing has been unfailing.

The sand drift comes from a western direction (not pushed by coastal sea currents) but impelled by the dynamic action of the waves that all year round hit the coast at an acute angle pushing the sand upward and towards the east, then receding on the inclined plane of the beach with a rip tide action. This continuous up and down motion with an eastern trend constitutes the eternal drift of sands along the coast, until the shoreline of Chiapas is reached; there, waves hitting the beach at a right angle push the sand up the beach and back to sea, without angle displacement.

The continuous action of the waves pushing the sands is the cause of the filling-up of the area between Beacon Hill and the far point of the breakwater referred to above, sometimes drifting so heavily into the outer harbor that it threatened to fill it up completely and close it to navigation. The same action has created coastal sand bars as in front of Juchitan, of such heights as to form lagoons with only small outlets for rivers and tidal currents.

At Salina Cruz Bay no strong tidal current is felt due to the small volume of water within the outer harbor and the basin; therefore, no open channel has ever been kept open by currents and the eternally drifting sand from the west has the tendency to fill up the outer harbor and block its entry.

The Administration of Free Ports faced this problem and, after considerable investigation arrived at the decision to erect at the land end of the western breakwater, on the same side of the drifting
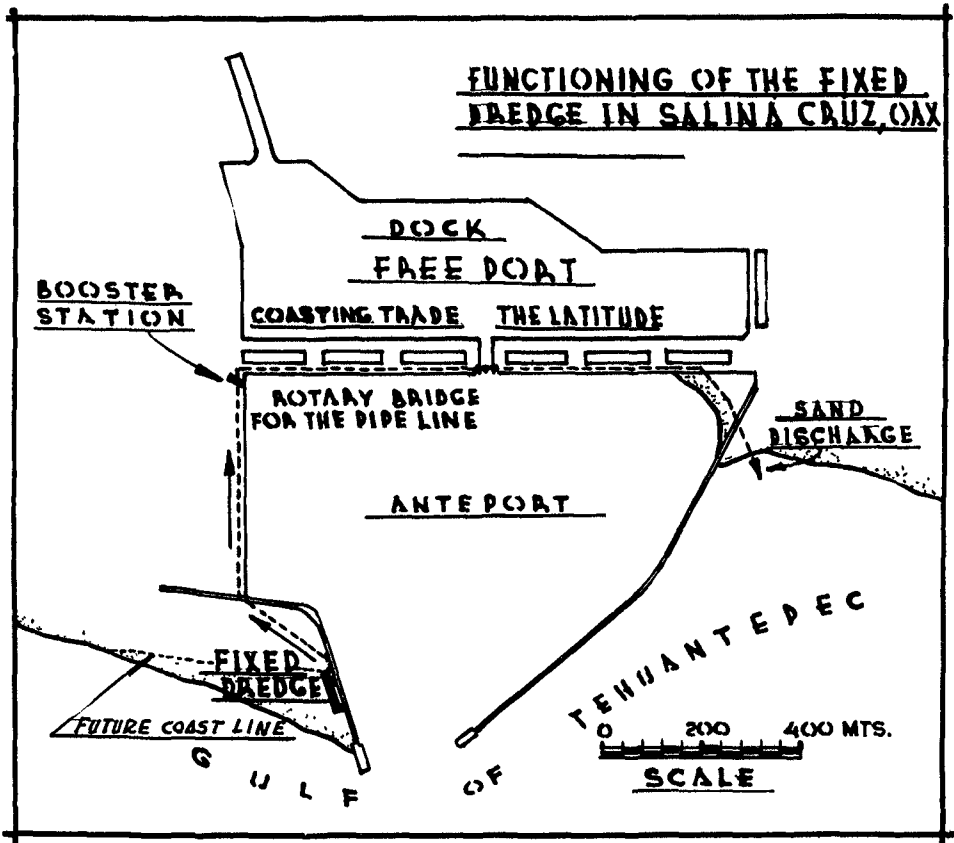
Fig. 1
Location of Salina Cruz stationary dredge and discharge pipe line.
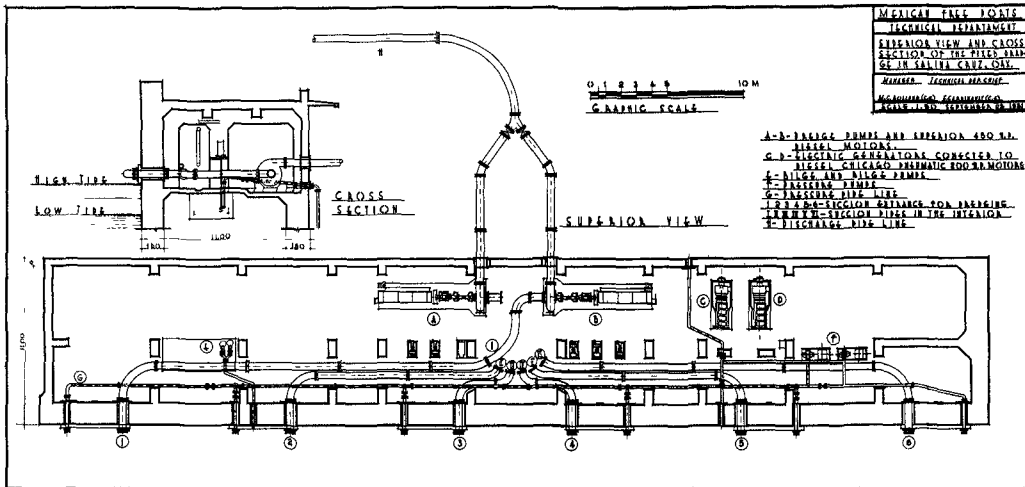
Fig. 2
Floor plan of machinery house.

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sand movement, a solid construction as illustrated by the enclosed drawing (Fig. 1) to house powerful suction pumps to carry away sand accumulated through a pipeline following the shore of the outer harbor, past the front of wharves, to the other side of the eastern breakwater where it is discharged on the beach, there to be washed further east by the sea.

Careful preliminary studies were prepared and checked by expert consultants brought from the United States; finally, sufficient appropriation was obtained from the Government to build three great blocks, each 65 ft. long by 39 ft. wide, aligned parallel to the breakwater. These great blocks with walls $4\frac{1}{2}$ ft. thick, with transversal partitions of the same thickness, were sunk, taking advantage of their own weight upon extraction of the sand collected in their interiors. This process is called "Chinese Well" and we understand this is the first time it has been applied in such a large scale in Mexico and, for that matter, in the Continent. The operation was carried out successfully, although not easily. The three blocks sank in perfect alignment with their bottoms resting 49 ft. below low tide.

On top of these blocks a superstructure was built and finished into a great 197 ft. long enclosed space to install standard dredging machinery, i.e. pumps, electric motors and generators, refrigeration pumps, vacuum pumps, etc. This machinery, however, has been installed in duplicate, to insure unfailing performance (Fig. 2). The pumps absorb sand on the sea front through six suction pipes that move fan shape by special swivel-joint connections, covering with their upward, downward and sideways movement, about 197 ft. or the entire length of the construction, digging out a great ditch 32 ft. deep where eastward drifting sand is deposited. It is a veritable sand trap.

Suction pipes are run by special derricks operated from a housing on top of the main building, each movement of the pipe, (upward, sideward and downward) originating in electric winches controlled by the operator sitting at a window between two of the suction pipes (Fig. 3). The sand then travels through a pipeline installed between the dredge and breakwater directly to where wharves begin, at the side of a substation which is in reality a duplicate of the main station, with identical motors and pumps. The pipeline then passes in front of warehouses and across the entrance of the eastern breakwater, thus transporting the sand for 6,600 ft. When traffic in the port becomes heavier, it shall be necessary to continue this pipeline from the substation, across firm land circling the basin, in order to avoid the swing bridge over the entrance.

This type of installation, with the machinery inside a building but doing actually the same work as a floating dredge does, is what has promoted us to call it a "stationary dredge". Its operation is quite simple and inexpensive since, to transport 35 cu. ft. of sand

(1 cubic meter) costs only twenty cents Mexican currency, from where the sand is pumped, to the other end of the pipeline, as against two to three pesos for the same volume, when done by a floating dredge.

Fig. 3
Preliminary arrangement to control moving sand

Personnel required for the operation is only a dredger, an electrician and assistants, also overseers.

The dredge does not let the sand pass on into the bay and so, when the installations begin to function normally, it will be time to dredge out the bay completely, wherein are now not less than 2 to 3 million cubic meters of sand that must be extracted before the port can be reasonably operated as such.

Without the stationary dredge, the extraction of sand with a floating dredge circulating in the outer harbor would be a never ending operation, as demonstrated by 40 years of experience. On the other hand, the stationary dredge planned by us and developed, thanks to the backing of the President, represents a new method of solving economically many problems represented by drifting sand. We expect, therefore, great international interest in this experiment considering that several other ports are in the same unfavorable condition and, if we are as successful as anticipated, we shall have contributed in a large measure to the betterment of world's production and trade.

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Fig. 4
Half section of swing bridge across the entrance to the basin,
in open position.

Fig. 5
Complete swing bridge in closed position.

SAND BY-PASSING PLANT AT SALINA CRUZ, MEXICO

This stationary dredge has cost ten million pesos of which six represent machinery from Werf Conrad, a Dutch factory specializing in dredges. This firm greatly aided us with their vast experience in the proper interpretation of our blue prints. The machinery was traded for Mexican sugar.

The engineering construction problem was ably carried out by "Obras Portuarias", S.A., with devotion, constancy and even sacrifice, in a relentless struggle against the sea, to achieve in the end success that deserves honorable mention. We should not leave out mention of another important cooperation, that of the Department of Communications to whom we owe a well-constructed, paved highway from the town of Salina Cruz to the stationary dredge, thus communicating the dry dock, oil terminal, and the dredge, all federal institutions highly important to the economy of the region.

STEEL PIPELINE AND SWING BRIDGE

The pipeline for transporting the sand through its 6,600 ft. length is made of steel 7 millimeters thick, flanged and jointed, and waterproofed. To cross the entrance from the outer harbor to the inner basin, a very light swing bridge was constructed where the pipeline acts as resistance axis, surrounded by a light iron web to add rigidity to the structure (Figs. 4 and 5). Two men, one on each side of the canal, handle each half of the bridge and tighten the central joint formed by a rubber section in the pipeline. The operation takes approximately ten minutes.

MACHINERY

The stationary dredge contains the following equipment:

1. Two "Superior" U.S. made diesel motors, each 450 H.P. to operate two dredging pumps, equipped with 18 in. pipes.

2. Two electric generators driven by 200 H.P. diesel motors to develop power required to operate all auxiliary machinery and winches.

3. Two duplex pumps, for refrigeration, lubrication, etc.

4. Two drain pumps.

5. Switchboard

Each pump functions by connecting a rubber, flexible suction tube to each of the six pipes coupled to the exterior of the machinery housing; these are coupled in turn, to the six suction tubes on the outside of the stationary dredge (Fig. 2). The water pressure pump

injects high pressure water separately to each suction pipe to agitate the sand and facilitate the suction process. On the $4\frac{1}{2}$ ft. thick block wall facing the sea and clamped by steel plates, six suction pipes are installed, moving upward, sideward and downwards, fanning thoroughly between pipes to dredge effectively the entire front of the stationary dredge.

Vertical and lateral movements of pipes are transmitted by cables from the top housing of the machinery building. Each suction pipe being equipped with three winches - there is a total of 18 winches. The operation of each suction pipe is controlled electrically by a dredge operator sitting at a window between two suction pipes.

In the stationary dredge the pump's two outlet pipes empty into the pipeline leading straight to the booster station at the end of the docks, crossing the access to the basin over the swing bridge (Figs. 4 and 5) and emptying, at a point at the head of the eastern breakwater, into the sea that carries the sand away to the east towards Chiapas.

BOOSTER STATION

The object of the booster station is to aid the movement of sand-laden water. It is equipped with two 450 H.P. motor driven pumps identical to those at the dredge. The station also is provided with cooling, fuel and refrigerating pumps, etc. The source of energy for these operations is the generators at the stationary dredge.

The 18 in. steel pipeline is open-installed in order to facilitate its inspection. It was first planned to cross the bay by means of an inverted syphon, but the suggestion was abandoned upon consideration of the possibility of it being subject to corrosion by salt water, precisely at a dangerous point like the channel.

SPUR DIKE

On the west side of the stationary dredge, at the foot of Beacon Hill, we are constructing a spur dike by the most economical standards, which we intend should hold back drifting sands from the west, to keep from reaching the dredge, thus saving considerable wear. Sometimes the bottom of the sand deposit forming at the head of the spur dike is less than 16 to 19 ft., due to the eastern drifting sand being pushed by the rip tide away from the deposit formed on the western side of the spur dike. This will be the only sand that requires dredging. We have calculated that a spur dike 1,600 to 2,000 ft. long can withhold sand for a period of 200 years but, when the stored sand forms a beach near the end of the spur dike, the rip tides force sand past the dike as explained above. This is the critical safety point of the spur dike against the drifting sand.

We need to determine, under actual working conditions, whether
the cost of the spur dike is cheaper than the cost of dredging stored
sand; if higher, it will be best to allow the sand to reach the dredge
and be pumped out by it. Experience will tell. It is advisable, how-
ever, to consider the construction of the dike even if it is costlier
than pumping sand, because it would save the machinery of the dredge
considerable wear. Even so, whether with dike and dredge or by dredge
alone, drifting sand will be stopped from entering the bay of Salina
Cruz.

PRE-OPERATION TO START THE DREDGE WORKING

Between the stationary dredge and the sea, a big volume of sand
stood where the waves have been pounding for ages. To start construc-
tion of the stationary dredge it was first necessary to raise a bank
reinforced by wooden piling. After the construction was finished and
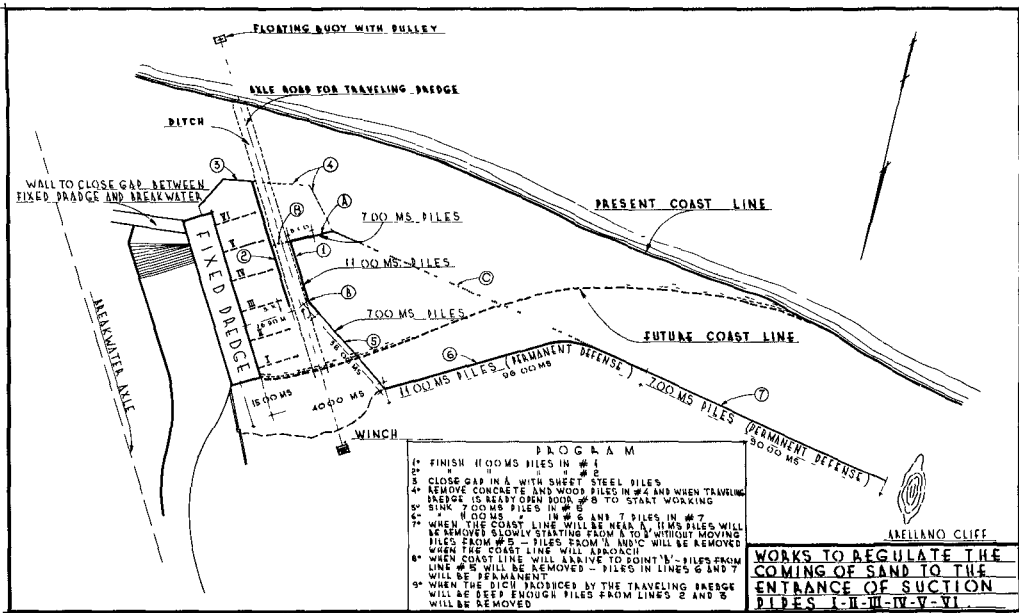
Fig. 6
Drawing showing position of piling installed to control impact
of sifting sand on the stationary dredge, considering the great
sand deposit created between the dredge and the ocean.

the machinery installed, tests were made to pump water and sand and,
even sinking of the suction pipes to extract sand from 32 ft. depths.
The next step was the opening of the lagoon, thus formed by the bank-
ing of sand, to the ocean front so as to sink the suction pipes into
the sea. This entailed a strenuous operation because each time the

opening was made, sand rushed into the lagoon in such quantities that it was filled up and the pipes stopped functioning. After several trials and arduous work, we finally had to construct passageways of reinforced concrete sheetpiling, in 23 and 32 ft. lengths, in front of the pipes, through which a drag scraper operates pulled by a winch and guided by steel cables passing over a pulley installed on a float anchored at sea (Fig. 6). This drag scraper or "travelling dredge" is actually digging a deep ditch across the beach in which drifting sands are trapped and carried to suction pipes number 1 and 2 (Fig. 2), then pumped off by the dredge. The sheetpiling will be displaced gradually, according to plan, but always leaving well defined accesses for sand until their greater volume is controlled to avoid fast precipitation into the lagoon, thereby throwing the suction pipes out of commission.

We were compelled to recourse to this operation because, if the large deposit of sand to be removed had had free access to the dredge, no machinery made would have been able to handle it. The machinery is calculated to remove normal volumes of sand drifting from the west; therefore, we had to hold back all accumulated sand and extract it gradually, at the same time that we dredge out other drifting sand.

The problem therefore of breaking out a deep ditch on the natural barrier built by the pounding sea has been a difficult one to solve, an arduous job to carry out and, the only solution to keep the sand from drifting further into the bay and to bring it within reach of the suction pipes. We understand this to be an entirely novel problem coming within the scope of the technique of maritime construction.

Chapter 16

RESEARCH ACTIVITIES OF THE BEACH EROSION BOARD

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The purpose of this paper is to describe the research program
of the Beach Erosion Board and to discuss the reasons which led to
the establishment of this program. Introductory statements are
given as to the functions of the Beach Erosion Board and the neces-
sity of research activities to support these functions.

The Beach Erosion Board - The Congress has given the Corps of
Engineers certain civil functions in adaition to its military dut-
ies; among these functions is the making of studies and recommendations
relative to the prevention of erosion by wave action of the shores
of the Atlantic, Pacific, Gulf, and Great Lakes. These studies may
be made directly for a Federal agency relative to the protection of
Government-owned land, or cooperatively with states, municipalities,
or other political sub-divisions.

The Beach Erosion Board was created some 20 years ago and since
that time has worked out solutions to many shore protection problems
in specific areas. Early in its work, however, the Board recognized
that beach processes were imperfectly understood and that the correct-
ness of the solutions to the problems at the specific beach areas
studied by the Board were subject to question as a result of this im-
perfect understanding. Accordingly the Board was given funds to make
general investigations in order to gain a more correct and thorough
understanding of beach processes and the effect on these processes
of various types of protective works, leading to the design of more
effective and more economical plans of shore protection and improve-
ment.

The Classification Table - In order to formalize the general in-
vestigation, or research, program of the Board it was found desirable
to classify the various factors (excluding economic factors) which
are believed to play a significant part in the understanding   and
solution of beach erosion problems. The resulting table is presented
on the following page.

TABLE 1

CLASSIFICATION OF FACTORS INVOLVED IN SOLUTIONS
OF SHORE PROTECTION PROBLEMS.

1. **Waves in Deep Water**
    A. Mechanics of Internal Movement
    B. Origin, Propagation, and Dimensions
    C. Effect of Certain Factors on Waves

2. **Waves in Shallow Water**
    A. Mechanics of Internal Movement
    B. Transformation Without Energy Loss
    C. Transformation With Energy Loss
    D. Origin, Propagation, and Dimensions

3. **Currents in Shallow Water**
    A. Internal Wave Currents
    B. Long-period Currents

4. **Factors Affecting Supply & Movement of Beach Material to Littoral Zone**
    A. Sources of Beach Material
    B. Rate of Transportation of Material to and from Littoral Zone
    C. Physical Characteristics of Beach Material Affecting Material Movement
    D. Long-period Water Level Fluctuations

5. **Significance of Natural Formations**
    A. Hydrographic Formations Higher than Surrounding Hydrography
    B. Hydrographic Formations Lower than Surrounding Hydrography
    C. Shore Line Formations
    D. Miscellaneous Formations

6. **Beach Processes**
    A. Mechanics of Material Transport in the Littoral Zone
    B. Rate and Result of Onshore-offshore Material Movement
    C. Rate and Result of Alongshore Drift.
    D. Creation and Alteration of Shore Forms
    E. Rate and Result of Wind Transport of Beach Material

7. **Functional Design and Effect of Man-made Structures**
    A. Structures Perpendicular to the Shore
    B. Structures Parallel to Shore
    C. Navigation Channels
    D. Artificial Fill
    E. Sand By-passing Plants

8. Structural Design of Man-made Structures
     A. Structures Perpendicular to the Shore
     B. Structures Parallel to Shore
     C. Sand By-passing Plants
     D. Resistance of Structural Materials

9. Supporting Investigations and Activities
     A. New Instrument Developments
     B. New Test Facilities
     C. New or Improved Test Methods and Procedures
     D. Preparation of Bibliographies and Reference Data

The factors listed in the Classification Table are not considered to
be of equal importance to the solution of shore protection problems
and additional factors may be added to the table in the future as more
light is gained on the subject. It can be seen that the Classification
Table has 9 categories with several headings under each category for
a total of 36 headings. Actually the 36 headings are further divided
into 99 additional subheadings; however, for the sake of brevity these
subheadings are not shown at this time, except those for Category 2,
"Waves in Shallow Water," which are shown on Table 2 as an example.

TABLE 2

2. WAVES IN SHALLOW WATER

A. Mechanics of Internal Movement
     1. Between deepwater and breaker zone
     2. At time of breaking
     3. After breaking
     4. Coalescence and interference of waves

B. Transformation Without Energy Loss
     1. By depth changes
     2. By refraction
     3. By diffraction
     4. By currents

C. Transformation With Energy Loss
     1. At breaking
     2. By bottom friction
     3. By bottom permeability
     4. By viscosity and turbulence
     5. By reflection

D. Origin, Propagation, and Dimensions
     1. Generation
     2. Forecasting including decay
     3. Opposing and cross winds
     4. Opposing and cross wave trains
     5. Statistical record accumulation
     6. Statistical record analysis

It is recognized that a classification table of factors relating to wave action on shores could be drawn up in forms other than the one presented. However, the table as shown was arrived at as being of practical application as a guide to the work of the Board.

The Priority Table - Once the Classification Table had been drawn up, a priority list or table was drafted. This Priority Table is, in effect, a statement, in order of priority, of the ten questions or problems most frequently encountered in specific shore protection studies for which adequate, quantitative answers are not generally available based on the present knowledge of beach processes. The Priority Table is shown below as Table 3. It is recognized that the order of priority as given is subject to question and might be re-arranged in significantly different order or some of the questions replaced by others depending on the past experience and immediate needs of the user. However, the questions and order given in Table 2 are the ones finally arrived at as best suited to the immediate needs of the Board. It is to be noted that the table is labeled as "Priority Table for Fiscal Year 1952;" this shows the table to be temporal and in practice it will be reviewed each spring for possible revision before drawing up the research program for the next fiscal year. The Classification Table presented as Table 1 is, on the other hand, considered to be a fairly permanent statement of the factors involved in the solution of shore protection problems.

TABLE 3

PRIORITY TABLE FOR FISCAL YEAR 1952

1. Rate and result of alongshore drift.
2. Functional design of shore-connected structures. (Groins and groin fields including length, height, and spacing)
3. Functional design of structures parallel to shore near the mean water line (bulkheads and seawalls) which special reference to criteria for setting crest heights.
4. Functional design of artificial fill as to elevation and width.
5. Functional design of sand by-passing plants.
6. Significance of inlets in shore processes.
7. Significance of horizontal breaks in shore line including headlands with special reference to littoral drift compartments.
8. Rate and result of wind transport of beach material.
9. Structural design of man-made structures.
10. Effect of offshore structures (particularly submerged breakwaters) on shore processes.

It can be seen that in general a complete answer to any one of the ten priority questions requires a quantitative understanding of several of the factors listed in the Priority Table. Thus a matching up of a given priority question with factors needed to answer the questions points up the directions in which the research program should move. A listing of the principal items involved in gaining a quantitative understanding of question #1 is shown in Table 4; the fifteen items shown appear among the ninety-nine subheadings of the Classification Table as described above and illustrated in part as Table 2

TABLE 4

STATEMENT OF PRINCIPAL FACTORS INVOLVED IN
ANSWER TO PRIORITY QUESTION #1

1. Rate and Result of Alongshore Drift.
  a. Wave forecasting, including decay, in deep water.
  b. Wave forecasting, including decay, in shallow water.
  c. Statistical record analysis in both deep and shallow water.
  d. Wave transformation by refraction.
  e. Statistical record accumulation in both deep and shallow
    water.
  f. Wave-generated currents in shallow water, including rips.
  g. Coalescence and interference of waves.
  h. Wave transformation by loss of energy from bottom friction
    and permeability.
  i. Wave transformation by loss of energy at breaking.
  j. Mechanics of littoral transport in the littoral zone.
  k. Physical characteristics of beach material affecting material
    movement.
  l. Supply of material to problem area from littoral deposits
    outside of area.
  m. Supply of material to beach from offshore area.
  n. Supply of material to beaches from inland areas by stream flow.
  o. Supply of material to beaches by marine life deposits.

  The Research Program - With the Priority Table and the Classifica-
tion Table at hand, a research program geared to the recognized needs
of the Board can be prepared. In preparing this program, the research
personnel available, the funds allocated to general investigations, and
the available test equipment are considered in addition to the priority
and classification tables. The resulting research program is presented
in Table 5 and is considered to represent a realistic program based on
the needs, funds, personnel and equipment available to the Board for
research purposes. Generally, the projects selected have been limited
sufficiently in scope to enable the results to be reported within a
year after the study is initiated. The relation of the projects to the
Classification Table and the Priority Table is indicated in that order
in parentheses following each project title. Table 5 appears on the
following page.

TABLE 5

BEACH EROSION BOARD RESEARCH PROGRAM FOR FY 1952

A.   Projects on which the collection of data has been completed and
     on which reports will be prepared in FY 1952.

    (1)   Study of quantity of sand in suspension in coastal waters
         (6A-1)
    (2)   Study of equilibrium profiles on beaches (4C-1)
    (3)   Study of model scale effect in movable-bed wave models (9C)
    (4)   Study of wave generation in inland waters (2D-1)
    (5)   Study of pressures developed by waves breaking against
         vertical structures (8B-9)
    (6)   Correlation of waves and alongshore currents (5B-1)
    (7)   Effect of Mission Bay jetties on adjacent beaches (7A-2)
    (8)   Preparation of charts showing effect of submerged break-
         waters on waves (7B-10)
    (9)   Preparation of reports based on Mission Bay field data
         (6B, C, D-6)
  (10)   Use of radio-active material for tracers in beach studies
         (9C)

B.   Projects scheduled to be undertaken and completed during FY 1952.

    (1)   Measurement of deep-water ocean waves by an airborne wave
         recorder (1B-1)
    (2)   Measurement of deep-water ocean wave by a spar-buoy wave
         gage (1B-1)
    (3)   Wave tank study of wave energy loss by bottom friction
         and permeability (2C-1)
    (4)   Wave tank study of wave run-up on shore structures (7B-1)
    (5)   Wave tank study of sand sorting due to wave action on
         sand beds (6B-1)
    (6)   Study of effect on beach profiles of varying wave periods
         (6B-4)
    (7)   Preparation of selected list of references pertaining to
         beach erosion (9D)
    (8)   Establishment and maintenance of wave recording stations
         in coastal waters. (This is a continuing project)
    (9)   Study of the stability of rubble mound structures in wave
         action.
  (10)   Study to determine the effective height of seawalls and
         bulkheads.
  (11)   Development of a settling velocity tube to provide a rapid
         means of analyzing sand under dynamic conditions.
  (12)   Study to improve the techniques of photographing tracers of
         the same density as water in wave action.
  (13)   Study of the economic life of shore protection structures -
         South Atlantic (Cape Charles, Va. to Florida Keys).
  (14)   Study of the composition, sorting, source and disposition
         of littoral material.

TABLE 5 (continued)

C.   Projects being conducted by outside agencies under contract to
       the Beach Erosion Board.

   (1)   Study of methods of analysis of wave records by electronic
           speech analyzers (1B-1)
   (2)   Study of methods of computing wave refraction over complex
           hydrography (2B-1)
   (3)   Development of method of computing refraction coefficients
           from one orthogonal (2B-1)
   (4)   Study to improve methods of forecasting alongshore currents
           (3B-1)
   (5)   Investigations of historic source and travel of sand found
           on existing beaches (4A-1)
   (6)   Geological study of San Nicolas Island beaches (4A-1)
   (7)   Study of submarine canyons as traps for littoral drift (5B-7)
   (8)   Development of instruments (9A)

   Of the studies listed under A and B above, the first 17 are being
carried on by the Research Division of the Board and the last 8 by the
Engineering Division. In addition to the 8 projects listed, the Eng-
ineering Division is also preparing a manual for the design of shore
protection structures. The manual will be based almost completely on
existing data and will be presented in five chapters as follows: De-
finitions; Functional Design; Structural Design; Economic Life; and
Design Analysis. It is planned to publish each chapter as a technical
memorandum as it is completed, and the completed manual as a technical
report. For this task the Engineering Division will be augmented tem-
porarily by obtaining the services of well-qualified personnel, re-
cruited from within the Corps of Engineers and educational institutions,who
have specialized in the various phases of work covered in the manual.

   General Investigations - In addition to the programs of the Research
Division and Engineering Division, the Reports and Publications Division
of the Board is compiling existing data on the shorelines of the United
States in such a form that it will be of maximum aid in the solution of
specific shore problems. These reports will be compiled on a regional
basis rather than a local basis and will serve to improve, simplify and
speed investigations of local problem areas. The existing data perti-
nent to shore processes in a selected shore region will be analyzed and
interpreted to the extent justified and the data and conclusions pub-
lished in the form of technical reports. The ultimate program contem-
plates the publication of twenty-three such reports covering the contin-
ental shores of the United States and three covering the territorial
possessions.

Assuming that the services of competent personnel can be obtained, eight such reports are to be compiled during the next two years. The eight regions selected for coverage are as follows:

1. The South Shore of Long Island
2. The New Jersey Shore from Sandy Hook to Cape May Point
3. The Peninsular Gulf Coast of Florida, from Cedar Keys to Cape Sable
4. The Gulf Coast of Texas
5. The Coast of Southern California from Pt. Fermin to the Mexican Border
6. The Coast of Southern California from Pt. Conception to Pt. Fermin
7. The Shores of Lake Michigan
8. The United States Shore of Lake Erie

Each report will consist of five chapters entitled:

1. **Geomorphology**
2. Littoral Forces
3. Littoral Materials
4. Littoral Measurements
5. Summary and Conclusions

The Research Division and Engineering Division of the Board will contribute to the preparation of the several chapters. Also, competent engineers and scientists both within and without the government service will be engaged, when possible and advisable, on a temporary basis to assist in the preparation of certain portions of the material.

Concluding Statement - The current research program of the Beach Erosion Board has been described. This entire program is pointed toward the accumulation and clarification of knowledge which will enable the Board to more effectively solve the various shore protection problems brought to it for advice and solution. The greater portion of the findings resulting from these investigations will be of interest to engineers and scientists engaged in professional activities dealing with wave action in coastal waters. The findings of the Board will be published in such a form as will make them available to all persons and agencies confronted with coastal engineering problems.

Part 3
SITE CRITERIA

Chapter 17

ENVIRONMENTAL CHARACTERISTICS OF SOME MAJOR TYPES OF HARBORS

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The operational requirements for an ideal harbor are (Chao Hwa et al. 1945, Stewart 1945):

a. Provision for shelter from the waves of the open sea and a limited fetch.
b. Provision for shelter from strong winds from all directions.
c. Depth enough for a large vessel to maneuver over an extensive area.
d. Good holding ground for anchoring.
e. A deep draft entrance from the sea.
f. A minimum of annual expense for dredging.
g. Moderate or small tidal range.
h. Moderate tidal currents.
i. Sufficient circulation to remove contaminants.
j. A low fouling rate and relative freedom from marine borers.
k. Freedom from seiches and surges.
l. Freedom from tidal bores.
m. Freedom from ice.
n. Freedom from fog.

The reasons for these requirements are clear enough so that no further explanation seems to be necessary. Most of these operational requirements can readily be translated into physical phenomena provided by land, i.e., by configuration of the coast or topography.

At this point the authors wish to state that the term environment as used in this paper, is meant to imply the natural physical environment only. The economic environment of harbors is an entirely different problem, beyond the scope of this paper and this conference.

Protection from waves of the open sea is a function of both isolation from the sea and the orientation of the entrance to the direction of the waves.

Protection from winds is generally provided by elevations of the land.

Depth and area are results of the topography of the submerged land, and a good holding ground for anchoring is controlled by the terrestrial sediments.

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The natural maintenance of a deep draft entrance is a more complex problem. It depends not only on the topography of the sea bottom which has to form a sufficiently deep trough-like depression but also on other factors. To name just a few, there has to be a mechanism or factor which keeps this depression from being filled in, such as absence of large amounts of suspended sediments carried in by rivers, or currents strong enough to carry such sediments into the sea, and also absence of long-shore currents which may build spits across the entrance of the harbor. As an example it may serve to point out that rivers draining large areas of un-consolidated or semi-consolidated material are apt to carry large amounts of sediments in suspension, while rivers draining areas of igneous or metamorphosed rock carry a relatively small silt load. The Mississippi discharges only about eight times the amount of water of the combined discharge of Chesapeake Bay rivers, however, the silt load of the Mississippi is about 750 times that of the silt load carried by the combined rivers entering the Bay.

The annual expense of dredging depends upon the afore-mentioned factors.

So far we have dealt with requirements which were controlled by features of the land. The next group is influenced by astronomical, oceanic, atmospheric, and biological forces.

Tidal range and resulting tidal currents, only partly dependent upon the configuration of the coast, are also a function of latitude, wind direction, and wind velocity. Storm tides are generally higher than spring tides for a given location. Triangular shaped indentations of the coast make for high tidal ranges because of the so-called funneling effect. Also their shape and size have an influence on their natural period of oscillation. In cases where this natural period coincides with the tidal cycle, as in the Bay of Fundy, the ranges are greatly increased. But these local ranges are only superimposed on the oceanic tides which have their highest ranges around 45 degrees north or south latitude and their lowest ranges near the equator and the poles. As to the tidal currents it is a fair generalization to state that the higher the tidal range the higher the velocity of the resulting tidal current.

Circulation of a harbor, or in other words elimination of sewage and industrial contaminants, is, according to the most commonly accepted theory, a function of fresh water inflow and tidal ranges. Whenever either one of these two factors fluctuates from the norm, the time element is changed.

Fouling is a complex problem. Fouling rate depends upon temperature, salinity, nutrients, light, and various other factors. Its importance lies in the fact that harbors with a high fouling rate may infect ships.

Marine borers attack all unprotected wooden structures and vessels, and are widely distributed throughout the world. Since the most common forms are highly tolerant of wide salinity and temperature ranges, little can be done to prevent their depredations short of chemical treatment or metallic protection of exposed wooden surfaces.

Seiches, surges, and tidal bores are rather rare phenomena, are not too well understood, and may be left out of a general discussion such as the present one.

The formation of ice is a function of latitude and presents a special problem which has to be studied locally.

It is rather difficult to generalize on fog within the scope of this paper. Here we deal with local physiographic phenomena only, because these are the ones which have major importance in the quality of a harbor. Fog rarely forms over a small area but usually covers areas considerably larger than a harbor. Of the various common types of fog, only two, steam fog and advection fog, are of importance over water surfaces. Steam fog occurs when colder air comes in contact with warmer water. Then, the saturation vapor pressure with respect to the water is higher than the saturation vapor pressure of the air. Hence, the water vapor that evaporates into the air will condense and form fog. This type occurs most frequently during the low sun period in high latitudes.

Advection fog forms when the air moves over a colder surface so that it is cooled from below by contact. This condition exists in mid-latitudes on the west side of continents during the high sun period when cold water wells up and provides a cold surface.

With the advent of radar, fog has become less of a hazard to the mariner.

We can hardly expect to find a location which will provide all the operational requirements here discussed, and it is the job of the harbor designer to make adjustments where nature has failed from man's point of view. However, certain physiographic types of configuration of the coast line are better suited than others as sites of harbors and it is the purpose of this paper to investigate to what degree these types approach the ideal.

Obviously not all types and variations thereof can be discussed here and the following major ones have been chosen and defined (Fenneman 1931 and 1938, Holmes 1945, Lobeck 1939, Moore 1949):

BAY - an indentation of the coast.

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a. Funnel-shaped Bay - a drowned mouth of a river.
b. Tectonic Bay - a bay formed by drowning of an orogenic belt where the structural grain is parallel to the coast. This type is generally confined to the shores of the Pacific Ocean.
c. Ria - a long, narrow, funnel-shaped bay caused by the submergence of a region of ridges and valleys, where these are not parallel to the coast. This type is generally confined to the shores of the Atlantic Ocean.
d. Fjord - a long, narrow, more or less steep-sided bay formed by glacial action.

RIVER - for the purpose of this paper, which deals only with salt water sites; Philadelphia, Pennsylvania, is defined as a river harbor because it has the aspects of a river despite the fact that it is still tidal.

DELTA - a fan-shaped alluvial tract formed at the mouth of a river.

a. Arcuate delta - a triangular-shaped delta.
b. Birds foot delta - a delta where the distributaries have built finger-like extensions into the sea.

LAGOON - A shallow stretch of water which is partly or completely separated from the sea by a narrow strip of land.

a. Atoll - a lagoon formed by a horseshoe or ring shaped coral reef.
b. Barrier reef lagoon - a lagoon separated from the sea by a coral reef.
c. Barrier bar lagoon - a lagoon separated from the sea by a sand bar.

Before we go into a more detailed examination of the enumerated physiographic features, it seems appropriate to discuss some of the parameters which apply to all of them.

One of these phenomena that seems to be important is the fact that any tidal inlet or indentation in the coast will have a flood side and an ebb side. This phenomenon is caused by the Coriolis' force deflecting any moving body to the right in the northern hemisphere and to the left in the southern hemisphere. The flood side, therefore, in the northern hemisphere is on the left hand side of an indentation in the coast looking seaward, and the ebb side is on the right hand side looking in the same direction. In the southern hemisphere the sides will be reversed.

At any given tidal inlet these sides have certain characteristics which seem worth mentioning. Mean sea level is higher on the flood side than on the ebb side and so are tidal ranges. Assuming that any tidal inlet has some sort of fresh water supply resulting in a net outflow, salinities are higher on the flood side where the sea water is brought in than on the ebb side where most of the fresh water flows out and, therefore, the fouling rate should be higher on the former. On the other hand, since the ebb side not only has to carry the waters brought in by the flood tide, but also the fresh water out to sea, the current velocities are higher on this side. The effect of the difference in current velocities is that coarser sediments will be found on the ebb side which may have bearing on the quality of the ground for anchoring. Furthermore, since most of the outflow takes place on the ebb side, it stands to reason that contaminants will be eliminated more readily on this side.

One might say that unless it is not advisable for other reasons, from a physical point of view, it seems more advantageous to construct a harbor on the ebb side of a tidal body of water rather than on the flood side.

However, if we hastily examine some of the major ports of the world, we find no clear cut relationship between ebb or flood side and port location. Economic factors such as easy location of terminals and access to the hinterland, overbalance the physical advantages apparently to be gained by location of a port on the ebb side of a tidal body of water.

This statement is presented here as a suggestion and as food for thought only. To the best of the knowledge of the authors this phenomenon has never been studied thoroughly and considerable research is necessary before more definite conclusions can be drawn.

FUNNEL-SHAPED ESTUARIES (Figure 1.)

From their nature as drowned mouths of rivers, these have certain characteristics. They were formed by rivers at a time when sea level stood lower than today and subsequently were invaded by the sea. Most of them still have a well-defined deep channel near their longitudinal axis representing the old river course. This channel is not necessarily continuous but in many cases is broken up into a series of elongated depressions. Funnel-shaped estuaries give good protection from waves provided that fetches are short enough to prevent generation of local waves, because the configuration of the coast line absorbs wave energy. On the other hand they are mostly located in coastal plains and do not

give protection from wind. They are generally deep enough for
maneuvering and have good holding ground for anchorage. A deep
draft entrance from the sea usually does not present a problem
and dredging requirements are, therefore, small. The same con-
figuration of the coast line, i.e., the funnel-shape, which gives
good protection from the waves, tends to increase tidal ranges
and resultant tidal currents. This increase, of course, makes
for good circulation and elimination of contaminants. In summary,
funnel-shaped estuaries offer good harbor sites except in latitudes
where winds and tidal ranges make their location impracticable.

TECTONIC BAYS (Figure 2.)

This type of bay is formed by drowning of an orogenic belt
where the structural grain is parallel to the coast. Examples
are found on the Pacific shore. San Francisco is an excellent
example of this type. They usually have a deep narrow entrance
from the sea and widen considerably behind it. They give good
protection from waves and also winds because of their mountain
fringe. They also permit ships to maneuver in a large area and
have good holding ground for anchorage. They require minimum
expense for dredging. Tidal ranges may, be fairly high and tidal
currents fairly strong. However, because of the narrow entrance,
circulation is rather poor and the elimination of contaminants,
therefore, is slow. Nevertheless, they offer very good sites
for harbors.

RIAS   (Figure 3a and 3b.)

Rias are long, funnel-shaped indentations in the coast caused
by the submergence of an orogenic belt where the trends are trans-
verse to the coast. They are mostly confined to the shores of the
Atlantic Ocean. The bay in which Brest is located is a good example
of this type. Rias are generally V-shaped, in cross section, quite
deep at their entrance, shallow landward and give good protection
from waves and winds except in cases where they are oriented in the
same direction as the prevailing wind. Area for maneuvering is
sufficient and in the inner part of the ria there is good holding
ground for anchoring. Tidal ranges are increased by the funnel-
shape and tidal currents are fairly strong, making for good circu-
lation and rapid elimination of contaminants. Generally speaking,
rias make very good sites for harbors.

FJORDS (Figure 4.)

Fjords are long, comparatively narrow, more or less steep
sided, U-shaped in cross section, submerged troughs which have
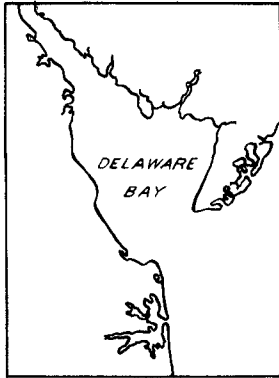
Fig. 1
Funnel shaped bay
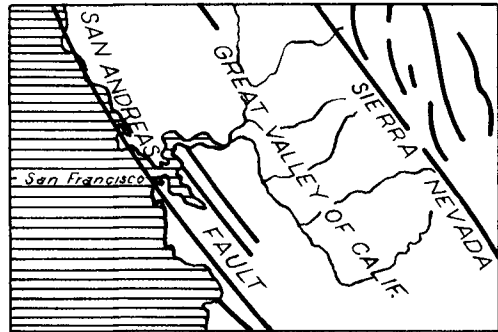
Fig. 2
Tectonic area of the
California coast
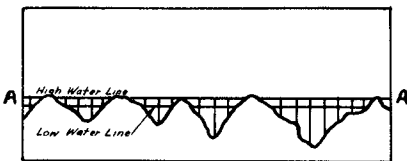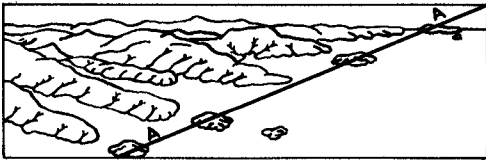
Fig. 3a
Ria-type coast line
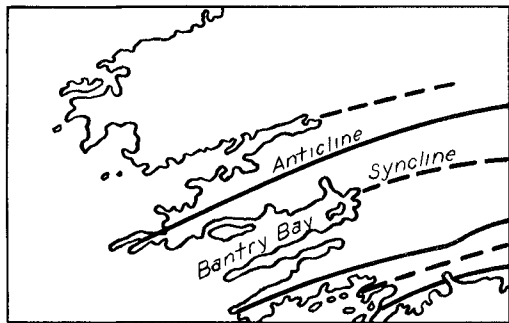
Fig. 3b
Rias of southwest Ireland

been formed by glacial action. Usually a fjord is quite deep, but becomes shallower towards its mouth, possibly because the glacier lost some of its powers of erosion as it melted or because a terminal moraine blocked the entrance. In other words, most fjords have a sill across the mouth, which may make entering hazardous. Protection from waves and winds is very good and so
• is the area for maneuvering; however, the bottom in fjords is mostly mud-covered, and holding ground for anchoring is poor because of the softness of the bottom as well as the fact that fjords are too deep and too much anchor chain has to be paid out. Tidal ranges and tidal currents do not present a problem. Fresh water inflow is usually small and for that reason as well as the fact that the sill at the entrance creates a pocket of rather stagnant water, circulation is poor and the elimination of contaminants very slow. Fjords make only fair sites for harbors.

TIDAL RIVERS (Figure 5.)

Tidal rivers usually make poor sites for harbors. Generally one oan say that on a straight portion of the river, assuming that all other requirements are met, harbors may be constructed on either side. If the harbor is to be located on the river bend, the outer side of the bend is more suited. Every river bend has a slip-off side that is the inner curve and an undercut side, the outer curve. It is always deeper on the undercut side where the stream actively erodes its bed.

Rivers give good protection from waves. Protection from winds is mostly poor because the tidal portions of a river usually flow through the coastal plains. They have generally only limited areas for maneuvering but offer good holding ground for anchorage. Deep draft entrances from the sea are not very common and expenses for dredging are quite high. High tidal ranges and strong tidal currents together with a strong net outflow make for rapid elimination of contaminants. Because of low salinities, fouling rates are low. Generally speaking, tidal rivers offer rather poor sites for harbors.

DELTAS (Figures 6 and 7.)

A delta is a fan-shaped alluvial tract formed at the mouth of a river when it deposits more solid material there than can be removed by tidal or other currents. In other words, if a river forms a delta it is a good indication that oceanic conditions

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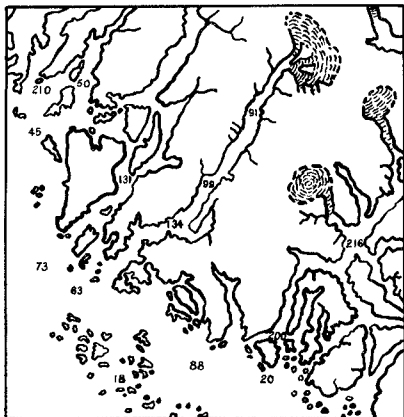
Fig. 4
Fjords
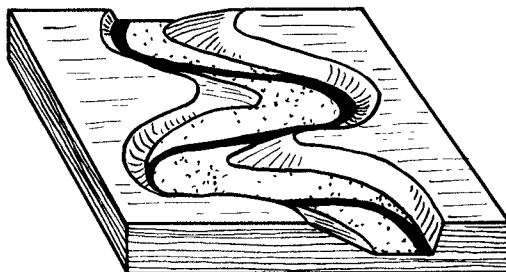
Fig. 5
River showing slip-off
and undercut sides
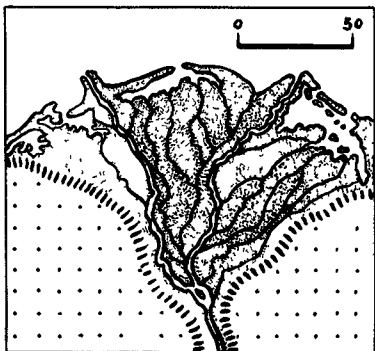
Fig. 6
Arcuate delta of the Nile river
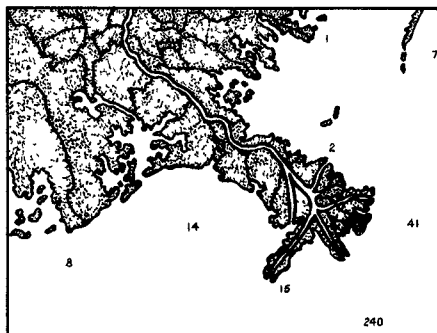
Fig. 7
Birds foot delta of the
Mississippi River

are rather calm. There are two types of deltas on which harbors have been located, the arcuate delta which the Nile has formed and the birdsfoot delta of which the Mississippi is a good example. Arcuate deltas make rather poor harbor sites. The distributaries give good protection from waves but protection from winds is poor because of the flat topography. The distributaries are also shallow, shift their courses, and require extensive and continuous dredging. Tidal ranges and tidal currents are small; however, a fairly steady river flow makes for quick elimination of contaminants. Conditions are better in the birdsfoot delta. Again, protection from waves is good, but protection from winds is poor. The finger-like distributaries flow between natural levees and are comparatively deep, requiring little expense for dredging. Tidal conditions and elimination of contaminants are the same as in the arcuate delta, with a small range but steady flow to produce rapid elimination.

LAGOONS (Figures 8 and 9.)

Lagoons are shallow stretches of water which are partially or completely separated from the sea by narrow strips of land. Two types of lagoons are interesting from the point of view of harbor construction: atoll and barrier reef lagoons, and barrier bar lagoons. In the case of atolls and barrier reef lagoons, separation from the sea is formed by coral growth and in the other case the separation is effected by a sand bar.

Both types offer fair protection from waves but poor protection from winds. Deep draft entrances are not very common and dredging requirements may be high. In shallow lagoons coral heads may also become navigational hazards and require removal. Atoll and barrier reef lagoons usually offer large areas for maneuvering, and holding ground for anchoring is good. Although tidal ranges are mostly small, and tidal currents weak, there are cases where they are fairly strong because of the large tidal prism plus the narrow passes. Elimination of contaminants is rather slow because of small tidal ranges and very little fresh water inflow and also because a pocket of stagnant water is formed inside the lagoon. In general, atoll and barrier reef lagoons make fairly good harbor sites.

Barrier bar lagoons usually have very small tidal ranges and very weak tidal currents. The area for maneuvering is restricted since this type of lagoon has a strong tendency to fill in; that is, it shallows in time, and barrier bars migrate slowly landward. Holding ground for anchoring is mostly good; however, because of the shallowness, dredging requirements are very high. Circulation is very weak and the elimination of contaminants very slow.
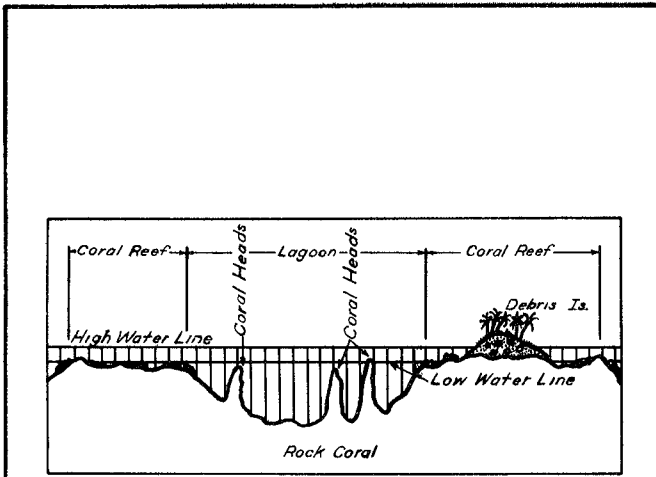
Fig. 8
Atolls and barrier reef lagoon
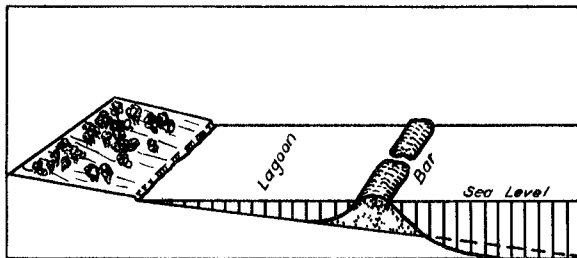
Fig. 9
Barrier bar lagoon

Venice is a good example of a harbor located in a barrier bar lagoon. It was a good harbor during the medieval ages, but lost its importance with the advent of larger ships with relatively deep draft.

In summary it may be said that the funnel-shaped estuarine bay (Delaware Bay), the tectonic bay (San Francisco), and the ria (Brest), afford very good harbor sites. Fjords (Trondheim) make only fair sites, and tidal rivers usually are poor for harbor locations. Arcuate deltas (the Nile) make rather poor harbors, but birdsfoot deltas (the Mississippi) are slightly better. Atoll and barrier reef lagoons make fairly good harbor sites. Barrier bar lagoons are generally poor because of the dredging requirements.

ACKNOWLEDGMENTS

REFERENCES

Chao Hwa, Chu Teng Kao, and Hsu Zan Ziang. (1945). A study of harbor design. Vols. I, II, III, IV. U.S. Waterways Experiment Station, Vicksburg, Miss.

Deasy, L. F. (1942). The harbors of Africa. Economic Geography. Vol. 18 (4), pp. 325-342.

Fenneman, N.M. (1931). Physiography of Western United States. McGraw-Hill, New York.

_____ (1938). Physiography of Eastern United States. McGraw-Hill, New York.

Holmes, A. (1945). Principles of physical geology. Ronald Press, N.Y.

Lobeck, A. K. (1939). Geomorphology. McGraw-Hill, New York.

Moore, W. G. (1949). A dictionary of geography. Penquin Book, England.

Stewart, John Q. (1945). Coasts, waves and weather for navigators. Boston.

U.S. Coast and Geodetic Survey. (1951). Current tables, Pacific Coast of North America.

_____ (1951). Tide tables, Europe and West Coast of Africa (including the Mediterranean Sea).

_____ (1951). Tide tables, West Coast, North and South America (including the Hawaiian Islands).

_____ (1951). Tide tables, Central and Western Pacific and Indian Ocean.

U. S. Navy Hydrographic Office. (1931). Sailing directions for the Northwest and North Coasts of Norway. H.O. Pub. 136.

_____ (1941). Sailing directions for the Bay of Biscay. H.O. Pub. 133.

Chapter 18

GULF HURRICANES AND THEIR EFFECTS ON THE TEXAS COAST

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The word "hurricane" is derived through the Spanish from a word of the extinct Indian aborigines of Haiti, meaning "evil spirit". I do not know whether the Indians who gave this kind of a disturbance its name are extinct because of the "evil spirit", but I am sure that it is a fitting name. Since the time of Columbus, there are records of hurricanes which have caused destruction and death in the West Indies and areas of Central and North America.

ORIGIN OF HURRICANES

The formation of these hurricanes in the northern hemisphere begins when air moves into a local low pressure area and the rotation of the earth directs the wind in a counter-clockwise spiral toward the center. As these disturbances expand into a tropical cyclone, they grow in size and velocity around the center, or "eye", which remains a relatively calm area of very low pressure.

The cyclone has two distinct movements - this rotary movement within the mass, and a progressive movement of the entire storm mass. These two movements are combined in much the same way as a rapidly spinning top which changes its location slowly.

BREEDING GROUNDS

Some of the hurricanes reaching the Texas Gulf Coast form in the region of the Atlantic Ocean lying generally north of the equator and south of the Cape Verde Islands between the northeast and southeast trade winds, and some form within the Caribbean Sea and the Gulf of Mexico. These regions have the supply of warm water vapors essential to the formation of hurricanes.

PATHS OF TRAVEL

The general pattern of hurricane paths runs westward or north-westward from the Atlantic Ocean into the Caribbean Sea and the Gulf of Mexico with a slow curving to the north which becomes more and more pronounced as higher latitudes are reached until the direction of travel is almost reversed and the storm either dissipates or heads

back into the Atlantic Ocean on a northeasterly course. This general
tendency, however, has a great many variations.

SEASONAL VARIATIONS

Hurricanes have occurred in the Gulf of Mexico only during the
5 month period from June through October. There is a decided variation
in the movement of hurricanes during this 5 month period. Early in the
season nearly all of them move from the western part of the Caribbean
Sea into the Gulf of Mexico, crossing the coast-line into Mexico or
the Gulf States. During August and September, and less frequently in
July and October, hurricanes develop on the Atlantic Ocean south of
the Cape Verde Islands and move in a westerly direction across the
Atlantic, some of them reaching the coasts of the southeastern states
before they curve to the northward and northeastward. Late in the
season they again originate largely in the western Caribbean Sea and
follow much the same course as storms from that region in June, except
that they are more likely to turn northward and northeastward in lower
latitudes, sweeping out over Florida or the Greater Antilles.

ABNORMAL TRACKS

In addition to these general variations there are so many exceptions
that the track of a single storm may bear little relation to the general
pattern. An example is the hurricane of October 30 - November 8, 1935,
known as the "Yankee Hurricane", which formed over the Atlantic Ocean
some 300 miles east of Bermuda. After approaching North Carolina, it
turned abruptly toward the Bahamas, then veered off to cross the tip
of Florida, describing a giant oval in the Gulf before dying out back
on Florida's west coast.

In early September of this year the large Hurricane EASY was
headed directly toward the island of Bermuda when the close approach
of the Hurricane FOX brought about a positive change in course, causing
EASY to miss Bermuda.

OCCURRENCES AND FREQUENCY

RECORDED OCCURRENCES

In the last 66 years, there have been 35 hurricanes that have
affected the Texas coast, distributed as follows: June - 6, July - 6,
August - 13, September - 7, October - 3.

Considering these 35 hurricanes actually recorded since 1886, it appears that some part or parts of the Texas coast will be struck by a hurricane on an average of about once every 2 years; however, cycles in frequency have brought more than half of the storms in a fourth of the time, leaving 39 of the 66 years hurricane free. Four hurricanes struck this coast in 1886, and in 5 other years 2 occurred.

The number of hurricanes experienced at each of six Texas ports is reasonably close, those to the north showing a somewhat higher rate. Port Lavaca, along the central coast, has experienced 2 less storms than any of the other 5 ports. The table below shows the number of times hurricane centers have passed within 50 miles of these 6 ports. The totals in this table exceed 35, since several hurricanes have affected 2 or more ports; however, most of those have approached the coast-line about perpendicularly.

Seaport areas affected	June	July	August	September	October	Totals
Brownsville	2	-	3	3	1	9
Corpus Christi	4	2	2	1	-	9
Port Lavaca	2	1	4	1	-	7
Freeport	1	3	4	2	1	11
Galveston	1	3	4	2	2	12
Port Arthur	1	2	5	1	1	10

Texas seaports may, on the average, expect to be affected by a hurricane about once in every 7 years; nevertheless, during the 66 years of record both the Port Arthur and Port Lavaca areas have enjoyed 19 successive hurricane seasons in which none were experienced; Galveston has had 18 years, Freeport 16 years, Corpus Christi 15 years, and Brownsville 14 years.

HISTORICAL OCCURRENCES

Information on the frequency of hurricanes before 1886 is fragmentary; however, there are records of 5 hurricanes in the last 5 years of the 15th century, 27 in the 16th century, 39 in the 17th century, 158 in the 18th century, 426 in the 19th century, and 166 for the 20th century through 1944. Many factors are cited as affecting the reliability of these figures.

There are accounts of severe gales in Galveston Bay in September 1766, and in September of 1818 four of Jean LaFitte's privateering vessels were washed ashore on Galveston Island. Eighteen other hurricanes are known to have struck the Texas coast from that time through 1885, some of them of very severe intensity.

HURRICANE MAGNITUDES

Winds with velocities of 75 miles per hour or higher are considered to be of hurricane force by the Beaufort Scale, established in 1805 and still in use. In general, Mr. Charles L. Mitchell of the U.S. Weather Bureau has classified tropical cyclones as being of hurricane intensity when they have a central atmospheric pressure of 29.00 inches or lower and are accompanied by winds near the center of more than 60 miles per hour. "Great hurricanes" are those in which the central pressure usually falls to or below 28.00 inches with the path of great damage about 50 miles wide. Some of the great hurricanes may cause damage in paths 600 miles wide. Only about one in ten hurricanes can be classed as "great hurricanes".

WIND VELOCITIES

There is good reason to believe that winds of an average rate of more than 150 miles per hour are sustained for five minutes and that gusts may reach as high as 250 miles per hour in the more violent storms. It appears that few, if any, hurricanes striking the Texas coast have reached these velocities. The extreme velocities of the 1900 and 1915 Galveston hurricanes were estimated at 120 miles per hour.

Since the atmospheric whirl north of the equator is counter-clockwise, there is added to the circular movement of the wind on the right-hand side of the storm, facing in the direction of its progress, the velocity of the general movement of the storm mass. Conversely, the winds on the left-hand side are decreased because of this action. The average velocities in the rear half are greater than the front half, making the right rear quadrant the most damaging.

STORM TIDES

The swells and waves generated by a hurricane travel directly forward from the right front segment of the storm at speeds calculated to be as much as 40 to 50 miles per hour and have been observed to precede the center of Gulf hurricanes by 600 miles. These rises, frequently referred to as "tidal waves", are not waves in any sense of the term, but are storm tides that exceed the undisturbed sea levels.

Both the topography of the Texas coast and low gravitational tides keep storm tides at or below 15 feet MSL as compared with a high of around 25 feet in the New England area.

DURATION

The average observed life of tropical storms in the Atlantic, Caribbean, and Gulf waters is about 9.5 days. Because many storms are probably unobserved in the early and later periods, it is likely that the average life is actually somewhat longer. August hurricanes have the longest observed average life of about 12 days; July and November storms have the shortest life, about 8 days. Several tropical storms have been tracked for 3 to 4 weeks and one, the 1900 Galveston hurricane, was tracked from the Mid-Atlantic west-northwestward to the Texas coast, thence north and northeastward across the United States to the Great Lakes, out over Newfoundland, back into the Atlantic Ocean, past Iceland and ultimately eastward into Siberia.

A hurricane with a diameter of 150 miles and a forward movement of 12 miles per hour would subject a specific locality to hurricane winds for at least 12 hours and to gusty squalls for something like 36 hours or longer.

PRECIPITATION

Heavy rain occurs in the forward half of the storm mass, considerably in advance of the center but very little falls in the rear half.

Heavy rainfall reflects radar impulses and may be satisfactorily observed through the use of specially designed radar equipment. Any unevenness in the downpours of the forward half of a hurricane, because of the rapid circular movement within the storm, appears on the radar screen as arcs from which the storm center can be determined with reasonable reliability. The Dow Chemical Company, as an adjunct to Weather Bureau activities, established at Freeport, Texas, a radar system to assist in minimizing hurricane damage to its extensive facilities located there. We are indebted to that company for the excellent motion picture record of their radar work during the approach of the 1949 hurricane to Freeport.

DESTRUCTIVE EFFECTS

DAMAGES

Perhaps the most destructive hurricanes in modern times have struck Calcutta, India. The 1864 storm drowned 50,000 persons and resulted in death to an additional 30,000 from disease. About 200,000 persons died as a result of the 1876 hurricane, approximately one-half of them drowning. Within the area of travel of

our Texas coast hurricanes, the "Great Hurricane" of October 1780 devastated the island of Barbados, sank an English fleet off Santa Lucia and killed 6,000 people on that island; sank 40 vessels in a French convoy off Martinique with a loss of 4,000 soldiers, killed 9,000 people on Martinique and left a path of devastation across other West Indian islands. The two islands of Martinique and Santa Lucia, where 15,000 people were killed, have an area of only about 600 square miles.

Estimates of property damage are not available from these storms; however, estimates range from $250 million to $330 million for the losses incurred in the September 1938 hurricane which hit the Long Island-New England area, killing about 600 people.

Over 6,600 lives have been lost and at least $136 million damage done in the Texas coast area since 1875, considering only the readily available information on the worst hurricanes, about 9 in all, during the period.

1900 GALVESTON HURRICANE

All of the destructive characteristics of hurricanes were brought to bear on Galveston during the September 8, 1900 storm. By five o'clock in the morning the tide was 4.5 feet above normal and had covered the lower sections of the city; by five p.m. a 9-foot tide had put water over the highest streets. The sloping beach, which under normal tides helped to break up the waves as they approached shore, no longer gave protection to the island, and when the maximum tide of 14.5 feet MSL came in the city's buildings were subjected to the full force of waves well over 20 feet high. Not only the buildings, but the land on which they were built, disappeared along a strip a block or more wide. Destruction of over 2,600 homes in a suburban area of 1,500 acres was complete.

Many of the total of 3,600 houses demolished were of sufficient structural strength to withstand the highest winds, but their foundations were undermined and they were battered with terrific force by the debris of less well built houses and other wreckage. In some places, however, debris gathered to form a protective barrier and was probably instrumental in saving other buildings from complete destruction. A large ocean-going steamer was torn from its moorings and carried several miles inland, requiring the construction of a canal to get it back to deep water.

More than 6,000 of the 38,000 population lost their lives and it is probable that the loss would have been much higher except

for early warnings which enabled thousands to leave the island before the highway and railway bridges to the mainland were submerged about noon, at least eight hours before the center of the storm arrived. Although the property damage of around $25 million has been greatly exceeded in more recent hurricanes, the loss of human lives was far greater than in any other North American hurricane.

In 1915 Galveston was hit again by a great hurricane; however, the developed part of the town had been raised in elevation and five miles of sea-wall along the Gulf shore had been constructed by Galveston County and the Corps of Engineers. Only 12 lives were lost and property damage was held to 4\frac{1}{2}$ million. There was extensive damage outside of Galveston, largely in the Freeport area, and the hurricane resulted in a total loss of 275 lives and property estimated at $50 million. This storm was of approximately equal intensity but of longer duration. It had a maximum storm tide of 12.7 feet MSL, 1.8 feet lower than the 1900 hurricane.

The difference between these and other Texas coast hurricanes is largely a matter of degree and extent of damages.

SHORE-LINE CHANGES

There have been evidences, ever since the settlement of the Texas Gulf Coast, of shore-line changes both above and below the water level. Under usual weather conditions these changes are so gradual that they become apparent only through observation over a period of several years. Changes are more obvious in the passes to the mainland through the chain of islands which makes up 4/5's of the 400-mile Texas coast-line.

Many times these passes are opened up by hurricanes only to be shoaled again in periods of normal weather. The 1949 hurricane cut 5 small openings in the Matagorda Peninsula between Port O'Connor and the Colorado River, and cut 2 small openings between the Colorado River and Brown Cedar Cut, but all of these are closing rapidly.

The Corpus Christi Pass between Mustang and Padre Islands is usually washed open by hurricanes, only to shoal up gradually afterwards.

A pass at Murdocks Landing was opened up across Padre Island by the 1933 hurricane. For 4 consecutive years the State Game and

Fish Commission dredged it, but it closed again within 4 months each time. In August 1945 local fishermen reopened it and a hurricane a few weeks later enlarged it; however, normal weather closed the pass once more.

None of the major passes are known to have been shoaled up by hurricanes since the jetties were built, although the jetties themselves have been damaged by the currents and waves which wash out stones and cause settlement or breaching through undermining and erosion. Under normal conditions all these major passes require periodic maintenance dredging.

One of the reasons for the scouring out of passes during disturbances is that the storm tides fill the bays and when the Gulf level recedes an appreciable head is left in the bays. The current from the bays to the Gulf, created by this head, is rapid enough to carry heavy loads of material from the channels.

On the other hand, some passes may be closed by hurricanes if the force, direction, point of attack and local conditions are favorable to this action. The Cedar Bayou Pass across Matagorda Island northeast of Rockport is fairly permanent, but was closed by the 1929 hurricane.

Some shoaling, but not enough to interrupt navigation, occurred in the Galveston Channel during the 1915 hurricane. The dimensions of the reach of the Gulf Intracoastal Waterway in the Matagorda and San Antonio Bays were reduced from 9 ft. x 100 ft. to about 6 ft. x 60 ft. by the 1942 hurricane when the material in Gulf-side spoil areas was carried back into the channel.

PROTECTIVE WORKS

For well over half of a century, protective works of various kinds have been undertaken by local interests and the Corps of Engineers. As an example, there are the jetties, from one to six miles in length, which have been built to keep open the five major passes through the coastal islands.

Some of the principal problems and a good part of the protective construction are found at Galveston, the only major Texas port which fronts directly on the Gulf. The construction of the south jetty in the period 1887-1897 reversed a tendency toward erosion and brought about an advance of about a mile in the shore-line of the east end. Further west along the Gulf side there was erosion up to about 300 feet; however, following construction of

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the sea-wall, after the 1900 hurricane, the beach built out 300 feet wide in places and it was possible to drive outside the wall for its full length. The 1915 storm scoured out the beach to a minimum depth of four feet and built up a bar parallel to the shore-line some 600 to 800 feet off shore. The natural accretion of normal weather periods moved the bar back into the shore-line. Various hurricanes up through 1942 threatened to scour out channels across low places in the island which would have seriously damaged the city, its harbor, its rail and highway outlets to the mainland, and its water supply lines. Additional protective works were provided and there is at this time a sea-wall extension job in progress.

In a following chapter a history of the Galveston Sea-wall by Mr. A. B. Davis of the Galveston District, Corps of Engineers is presented. I would like to point out the cooperative manner in which local interests and the Corps of Engineers are working on these improvements. We made recommendations to Congress for a sea-wall extension project in which both local and Federal governments were to participate. In view of current conditions, Congress made no appropriation to carry out the Federal participation at this time. Galveston County furnished its share of funds, enabling the Corps of Engineers to go ahead with construction of another mile of sea-wall.

<div align="center">REFERENCE</div>

Tannehill, I.R. (1944). Hurricanes; Princeton University Press, Princeton, New Jersey, 5th edition, 269 p.

Chapter 19

PERTINENT FACTORS IN THE PROTECTION
OF THE GULF COAST

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Protection of the U. S. shores of the Gulf of Mexico against
erosion or damage by waves, currents, or other littoral forces involves
unique physical and economic conditions that make this area quite dif-
ferent from other shore regions. It is the purpose of this paper to
discuss these conditions and the factors that contribute to them.

ECONOMIC CONDITIONS

It is a poorly recognized fact that there is only one major con-
centration of development and population located immediately on the
Gulf coast of the U. S., namely Galveston, Texas. Other major concen-
trations, such as Tampa, Mobile, New Orleans, Corpus Christi, (see
Figure 1) are located inland from the shore in areas relatively well-
sheltered from the open Gulf. Numerous small concentrations are bunched
on the shore in a few areas, such as the Naples to Tarpon Springs area
in Florida, and the Pascagoula-Biloxi-Gulfport-Pass Christian recrea-
tional center of Mississippi. These areas are primarily recreational
in character and derive their economic importance from this usage. The
concentration of population is relatively low on an annual basis but
reaches high values during "the season" which may last five months. In
these areas the most important single element in the economy is the
existence of a beach whose character is pleasing to the population seek-
ing recreation at the particular resort. The loss of or reduction to a
less satisfactory condition of any one of these recreational beaches
would be serious to the local property owners and resort business pro-
prietors, but probably would not be of sufficient national significance
to be noticed.

Long stretches of the Gulf coast are essentially wilderness areas;
such as the extensive barrier islands of the Texas coast and the isolated
chandeleur chain off the Mississippi and Louisiana coast, or the vast
marshlands of western Louisiana. At the present time the economic sig-
nificance of these areas, constituting well over half the total coast-
line must be considered as slight.

Under these circumstances the problem of the economic worth of these
lands and the protection of their shores needs to be examined carefully.
In at least one case submitted to analysis, that of Anna Maria and Long-
boat Keys located just south of the Tampa Bay entrance, the Beach Erosion
Board could find no economic justification for protection of the area and
reported its opinion that no public interest was involved in the protec-
tion. It was recognized however that local property owners might never-
theless desire to protect their property in spite of the lack of economic
justification and methods of achieving protection were outlined in the

Board's report. Particularly in regard to the Gulf coast one must guard against blind acceptance of the usual view of the property owner that the mere existence of the beach or shore is adequate reason for maintaining the existence of that beach or shore by protective measures. One cannot help but subscribe to the concept that, under present conditions of use and development, protection of major portions of the Gulf coast cannot be justified on an economic basis. In fact, probably the best treatment for the area in general is to leave it in its natural condition and confine development to regions that are known to be relatively stable, planning the development with adequate allowance for the free play of natural forces and the normal processes of development of unprotected shores. It is to be noted that the development of the Gulf coast whether by chance or design, now follows this latter pattern, with some notable exceptions.

PHYSICAL CONDITIONS

The uniqueness of the Gulf environment in respect to shore protection lies as much in its physical attributes as in its economic development. In broad terms the Gulf of Mexico is a roughly elliptical basin of quite shallow rim and relatively flat offshore slopes leading to a central portion with depths in excess of 6,000 feet (See Figure 1). The Gulf is almost landlocked, communicating with the Caribbean by the deep Yucatan Channel and with the Atlantic by the Straits of Florida. There is a well-developed continental shelf in the Gulf with a continental slope leading to a central deep of roughly the same outline as the land border of the Gulf. The dominant morphologic feature of the Gulf is the Mississippi Delta, which seems to lie upon and cross the continental shelf. The land borders of the northern Gulf are very low-lying; with the 100 ft. elevation contour located from twenty-five to more than a hundred miles inland. The principal topographic characteristic of the landscape is its monotonous flatness. The shore is principally of the barrier-beach type, where extensive sand, or sand and shell, beaches front lagoon or marsh areas that stretch for miles along the coast.

In any locality the problem of shore protection is, in its broadest terms, a matter of maintaining some selected land-water boundary. This is attempted in a variety of fashions, ranging from massive sea walls to the repeated artificial deposit of sand as needed to maintain a favorable balance of sand supply on a protective beach. The various factors controlling the protection method to be employed have been discussed in other articles and will not be repeated here, except to recall that in general one must have available adequate knowledge for the locality concerned of water levels, or tides; wave action; currents; storms; the sources of material for the beaches; and any unusual natural forces, such as hurricanes. Some of these factors as they apply in the Gulf area will be discussed.

Fig. 2. Hurricane tide frequency.
Based on observations 1900-1951
at various locations.

Fig. 4. Typical profiles of
Gulf bottom.

Fig. 1. The Gulf of Mexico. Depths
and elevations are in feet. Dotted
lines represent locations of pro-
files shown on Fig. 4.

Fig. 3. Significant periods and
heights of waves at Humble Plat-
form 8 miles offshore Grand Isle,
La. 1948-1949.

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TIDES - WATER ELEVATIONS

The periodic tides in the Gulf of Mexico are small, varying from about 0.7 ft. to 1.8 ft. mean range, and about 0.9 ft. to 4.0 ft. spring range. Extreme tides, exclusive of hurricane epochs, range from 1.5 to 3 ft. below mean low water, and 3 to 6 ft. above mean low water. In the region from about Cape San Blas, Fla., to about the U. S. - Mexican border the tides are irregular and mostly diurnal, i.e. there is one high and one low water daily; elsewhere on the U. S. Gulf coast there are two high and low tides daily.

Although the tidal fluctuation in water elevation is usually low, variations in water elevation of about 4 ft. below and 4 ft. above tidal stage due to the effects of wind occur sufficiently often to be considered of the same importance as tidal fluctuations, and are the controlling factor in design rather than tides.

Variations in water elevation due to hurricane effects along the Gulf coast are large and may occur with sufficient frequency to justify their use in designing protective works.

The elevations reached on the coast by the Gulf waters and the frequency of their occurrence determine the elevations to which protective structures or works must be built to prevent inundation of the protected areas. In addition consideration must be given to the wave heights that may occur coincidentally with high water elevations in arriving at the design elevation of the protective works. Figure 2 is a frequency curve of water elevations prepared from hurricane tide records for the Gulf Coast, and shows also the record high water elevations for several typical locations. Data of this type prepared for various localities being considered for protection allows an evaluation to be made of the minimum design elevation of works to prevent inundation and provides part of the basic data required for economic analysis of other-than-minimum works. In the absence of more complete data the information in Figure 2 can be used for design purposes for any of the U. S. Gulf coastal locations.

Minimum works should be considered as being those whose top elevations will prevent overtopping during frequently occurring water stages under non-hurricane conditions; in the Gulf area these top elevations are of the order of 4 to 6 ft. above mean low water.

WAVES

Knowledge of wave action is important in shore protection design for several reasons. In the case of structures, such as seawalls and bulkheads, an increase in the height of the structure commensurate to the wave height over that required for simple inundation protection against rising water levels in necessary. If there is material available to build or maintain a protective beach the direction, height, and frequency of occurrence of waves is of value in evaluating littoral drift characteristics, designing the protective beach (particularly its top elevation and

width), designing the groin profile if groins are to be employed, and in the structural design of the groin. Other uses may exist.

Relatively little is known about wave action nearshore in the Gulf of Mexico. For a short time the Beach Erosion Board and the Humble Oil organization operated jointly a wave measuring apparatus on a Humble drilling platform about 8 miles offshore of Grand Isle, La. The results obtained are summarized, by months, as shown on Figure 3. It will be noted that the nature of the wave action in this location is quite uniform, with periods averaging about 5 seconds, and wave heights about 1 foot, throughout the year. The hurricane months of July through October show some waves of up to about 7 feet height and 8-9 second period but they are of infrequent occurrence and short duration.

In comparison and as additional data the reader is referred to Sea and Swell Charts - Northeastern Pacific Ocean (U.S. Navy) which presents in detail the results of ship observation of sea and swell in the open Gulf over a number of years. The data is not reproduced here because of its volume. In summary it shows that on the average the Gulf is calm 50 to 60 percent of the time during May through August, and about 30 percent of the time during the remainder of the year. When waves exist in the northern half of the Gulf they travel from the east or southeast most of the time, and range from 1 to 12 ft. in height predominantly. No information on wave period is given on these charts.

It is of particular interest to note that the Florida West Coast is peculiarly fortunate in respect to wave action in that it is sheltered almost completely from high, destructive swell and is subject, for all practical purposes, only to locally generated and hurricane wave action.

Although detailed information on wave action in the northern part of the Gulf is deficient for analysis purposes the topographic character of the Gulf bottom and its effect in modifying waves by refraction make it possible to specify some important wave characteristics at the shore. Figure 4 shows several typical profiles of the Gulf bottom at the locations shown on Figure 1.

If it be considered that the maximum waves to be expected in the Gulf result from hurricane action some idea of their magnitude may be derived from the Sverdrup-Munk wave prediction method. It may be assumed that under these conditions maximum wind velocities of 60 knots blow over fetches of about 150 miles for durations not exceeding 20 hours. These conditions can produce maximum waves of 55 feet height and about 11 seconds period in deep water. This wave can transport bottom material in depths of about 500 ft. and will break in depths of the order of 70 ft., (say 12 fathoms). Referring to Figure 1 it is seen that the bottom can be affected as far offshore as about 150 miles, and that the maximum waves will break as far as 40 miles offshore.

A moment's thought leads to the belief that probably the most useful criterion of wave height to be expected at the shore is the water depth existing there under hurricane conditions. Reference to Figure 2 then leads to the belief that the probable maximum wave height to be expected at the shore near Tampa would be that of a wave breaking in about 11.6 ft. depth, or say 9 ft., at Galveston in about 14.5 ft. depth, or say 11 ft.

The probability that any particular Gulf coast location may be subject to maximum hurricane tide and wave action is illustrated by Figure 5, which shows the paths of hurricanes of record by month of occurrence. It is seen that maximum conditions, which are associated with passage of the hurricane over the area, can be expected to occur at any U. S. Gulf coast location. The maximum conditions to be expected are, as noted above, dependent upon location and hurricane intensity.

MATERIAL SUPPLY TO GULF SHORE

The most striking feature of the U. S. Gulf coast to an observer interested in shore protection is the seemingly unlimited quantities of beach material in the area, as evidenced by the miles upon miles of wide, sandy beaches. Even in the western Louisiana marsh area one finds excellent beaches. Associated with this profusion of sandy shore is the immense delta of the Mississippi River, spreading octopus-like as a "sedimentary section some 30,000 feet thick, extending coastwise for over a thousand miles, with a width, in its landward limb, at places exceeding 350 miles" (Russell, 1940). One can find nearly as many opinions as to the extent and character of the Mississippi delta as one can find writers. It is sufficient for the purposes of this paper to accept the apparently incontrovertible opinion that the sediments of the northern Gulf of Mexico coast are very closely related to the sediments carried to the Gulf by the Mississippi.

Study of the wealth of geologic data on the Mississippi delta has led to the conclusion that the Gulf coast geologic record is "one of continuous sedimentation, southward migrating shore lines, interrupted only slightly by minor advances and retreats of the sea and structural deformation related to sedimentary loading - - ." (R.J. Russell, ibid)

Contributing factors to this situation have been evaluated variously by different authors, some of whom have argued to variant conclusions from the same basic data. The truth does not seem to have been established, and the figures or quantities stated hereinafter must be regarded then as approximations to the truth.

The true nature of the sediment burden brought to the Mississippi is not known. There is, however, abundant evidence that the burden has included significant amounts of gravel, sand, and fine-grained soils down to colloidal clays. The materials are carried in solution, suspension, colloidal dispersion, and bed load. Russell seems to attribute
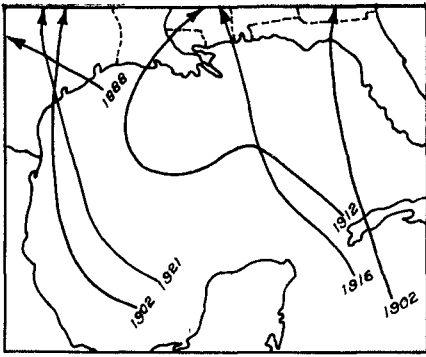
JUNE
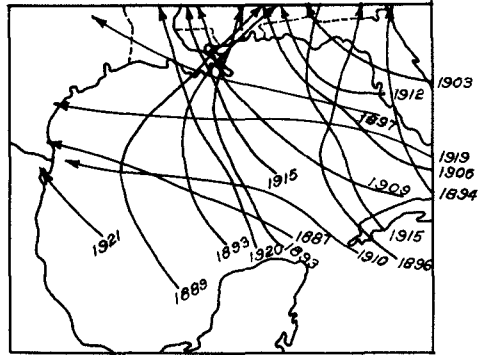
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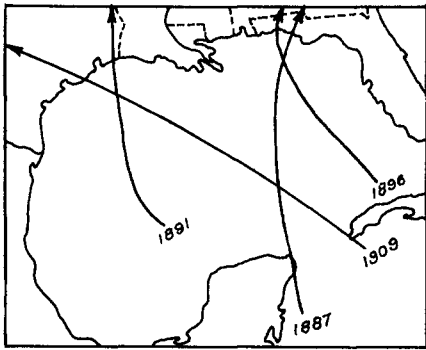
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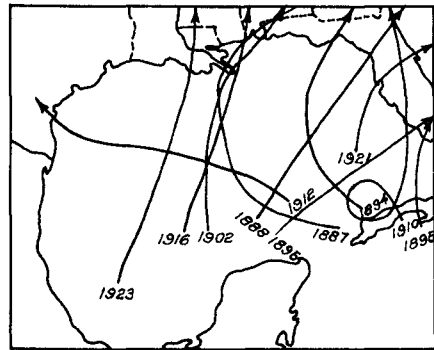
OCTOBER
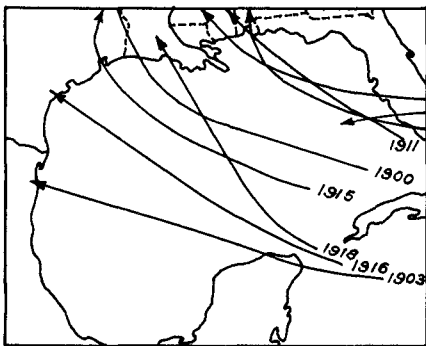
AUGUST

Fig. 5

Hurricane paths of record, shown
by month of occurrence. The dates
on the paths are the years of
occurrence.

the sandy shores of the Gulf to selective sorting of the river sediments, so that coarse materials are carried shoreward and finer materials out toward the sea, following a concept of De La Beche expressed in 1853.

Estimates of the quantity of material brought to the Gulf by the Mississippi vary between quite wide limits with a probable minimum average of 2,000,000 tons per day, or about 730,000,000 tons annually.

The age of the delta has been estimated variously as being from about 4,000 years minimum to about 140,000 years maximum, the median of the estimates being about 40,000 years. Thus there is fair evidence that the Gulf basin has received at least about 29,000,000 million tons of sediment for distribution over its extent and around its shoreline. Little would be gained by additional manipulation of these figures. I, at least, am prepared to conclude that a principal source of material for the natural building and maintenance of the extensive sand beaches of the Gulf area has been and is now the sediment brought to the area by the Mississippi River.

Concurrently, however, I am forced to consider that distribution of the material from its area of introduction to the shores must take place primarily during storm or hurricane conditions, the only times when there appear to be available wave or current forces adequate to transport large quantities of material. During other intervals the wave action seems to be, from available evidence, inadequate to perform the transportation role required.

There are, of course, many rivers other than the Mississippi which ultimately reach the Gulf. The principal to be considered as possible sources of material are the Alabama – Tombigbee River complex, the Caleasieu, the Sabine-Neches complex, the Trinity, the Brazos, the Colorado, the Lavaca, the Guadalupe, and the Rio Grande. Among these only the Brazos and the Rio grande enter the Gulf directly, all the others emptying into lakes, bays, or lagoons separated from the Gulf by barriers, and which serve as immense settling basins.

The Brazos has a well-developed delta in the Gulf near Freeport and therefore undoubtedly contributes some material to the building and maintenance of Gulf beaches. The Rio Grande, on the contrary and in spite of its reputation as a notorious debris carrier, appears to have no Gulf delta associated with its entrance. The meandering character of the stream and the extensive marsh development at its mouth, coupled with lack of a delta lead to the belief that the debris load is dropped before reaching the Gulf. Its present contribution of shore material probably is limited to whatever quantities are derived from its alluvial fan.

PERTINENT FACTORS IN THE PROTECTION OF THE GULF COAST

SUMMARY

In summary the Gulf coast appears to be unique, in respect to protection of its U. S. shore, by reason of its physical environment and economic development. The Galveston area is the sole major concentration of development and population in an exposed location on the Gulf shore. From an economic point of view a major part of the Gulf shore must be considered as being presently not sufficiently developed to warrant protection against natural erosion agents.

The physical environment of the Gulf requires that protection methods employ as primary design elements the effects of hurricanes, the influence of the flat offshore bottom slopes, and the predominance of the Mississippe River as the prime source of beach material.

The uniqueness of the Gulf situation does not however result in simplification of the protection problem. In fact, until much more is known about the details of the elements mentioned the Gulf coast will be one of the difficult problem areas for coastal engineering.

REFERENCES

U. S. Navy, Hydrographic Office (1944). Sea and Swell Charts, Northeastern Pacific Ocean (includes Gulf of Mexico). H.O. Misc. Publ. No. 10712-D.

Russell, R. J. (1940). Quaternary History of Louisiana, Bulletin Geological Society of America, Vol. 51, No. 8, pp. 1199-1234.

De La Beche, H. T. (1853). The Geological Observer, London, 2nd Ed.

Barton, D. C. (1930). Deltaic coastal plain of southeastern Texas, Bull. Geol. Soc. Am., Vol. 41, pp. 359-382.

U. S. Coast & Geodetic Survey (1951). Tide and Current Tables, Atlantic Ocean.

Army Air Forces, Weather Information Branch (1943). Hurricanes affecting the Atlantic Coast, the Gulf Coast, and the Southern California Coast of the United States, Report No. 636.

COASTAL ENGINEERING

Chapter 20

HARBOR AND COASTAL PROBLEMS ON THE EAST GULF COAST

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DESCRIPTION

GENERAL FEATURES

The "East Gulf Coast" discussed herein embraces the coast of the Gulf of Mexico from Cape Sable, Florida, generally northerly and westerly to the Rigolets, Louisiana (See Figure 1). So far as concerns Federal waterway improvements, the section is under the jurisdiction of the South Atlantic Division, Corps of Engineers, U. S. Army, Atlanta, Ga. That section in Florida as far northward as the mouth of the Aucilla River is administered by the District Engineer, Jacksonville, Fla.; thence westerly to the Rigolets, by the District Engineer, Mobile, Ala.

With few exceptions, the mainland along that section of coast is monotonously low and flat. Except for the 180-mile stretch between Anclote Keys, Fla., and Apalachee Bay, Fla., the coast is generally characterized by a line of low, narrow, sandy offshore islands and peninsulas, which enclose and protect a chain of shallow coastal bays, sounds, and lagoons. Between Anclote Keys and Apalachee Bay, the Gulf bottom to 30 or 40 miles offshore is generally a soft limestone rock covered by a thin layer of sand; barrier islands are absent, and the bottom is very flat, 3-foot depth being reached from 1 to 4 miles offshore. Elsewhere the Gulf bottom is generally sandy, and slopes somewhat more rapidly into deeper water.

TIDES

Lunar tidal ranges are moderate, the mean range being generally between 2 and $3\frac{1}{2}$ feet between Cape Sable and Apalachee Bay, and thence less than 2 feet westward to the Rigolets. A peculiarity of the lunar tides along that reach is that they change gradually and progressively from a normal semi-diurnal type at Cape Sable to a diurnal type at and west of St. Joseph Bay, Fla. Hurricane winds build up tides to an elevation of 6 feet or more above normal; northerly gales, occurring usually in winter, frequently lower the water surface by 1 or 2 feet, and occasionally by as much as 4 feet, correspondingly decreasing the depths in navigable coastal waterways below those based on mean low water.

WINDS AND STORMS

Between March and September, the prevailing winds along the east Gulf coast blow from the easterly to southerly quadrant and are usually moderate. During the late fall and winter, however, several polar air masses from the North American continent penetrate to the Gulf coast each year, bringing northerly winds up to over 50 miles an hour, locally known as "northers". Hurricanes, traveling usually northwesterly across Cuba and the southern tip of Florida, veer to the north and northeast and pass over the east Gulf coast, usually during August, September, or October. Winds to the east of

THE EAST GULF COAST

Fig. 1.

MOBILE   CO

SALINITY IN PARTS PER MILLION

RANGE	1	2	3	4	5
MILES	31	71	138	205	262
TOP	4,450	2,800	330	30	11
MIDDEPTH	5,900	4,450	360	32	12
BOTTOM	12,600	12,600	10,100	2,600	12

River Discharge - 10,700 cfs

SALINITY STUDY
MOBILE HARBOR
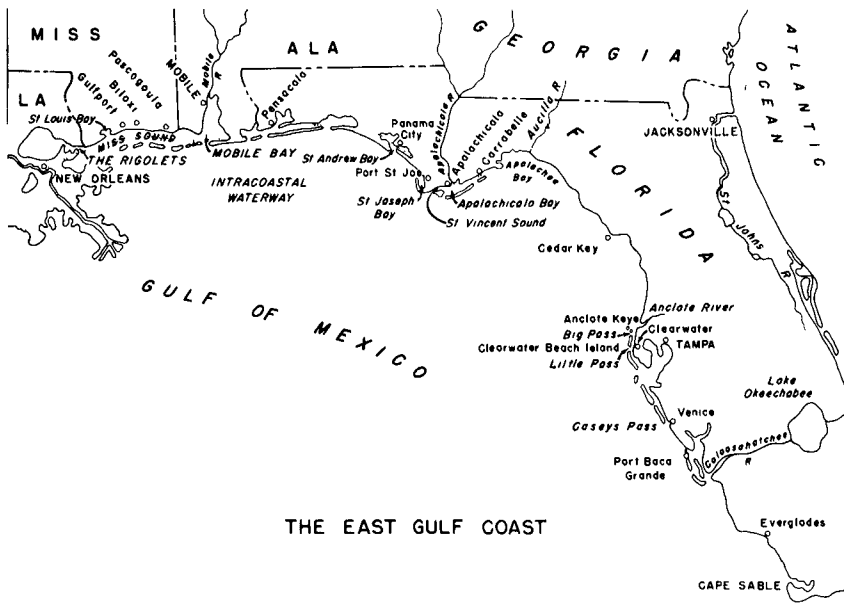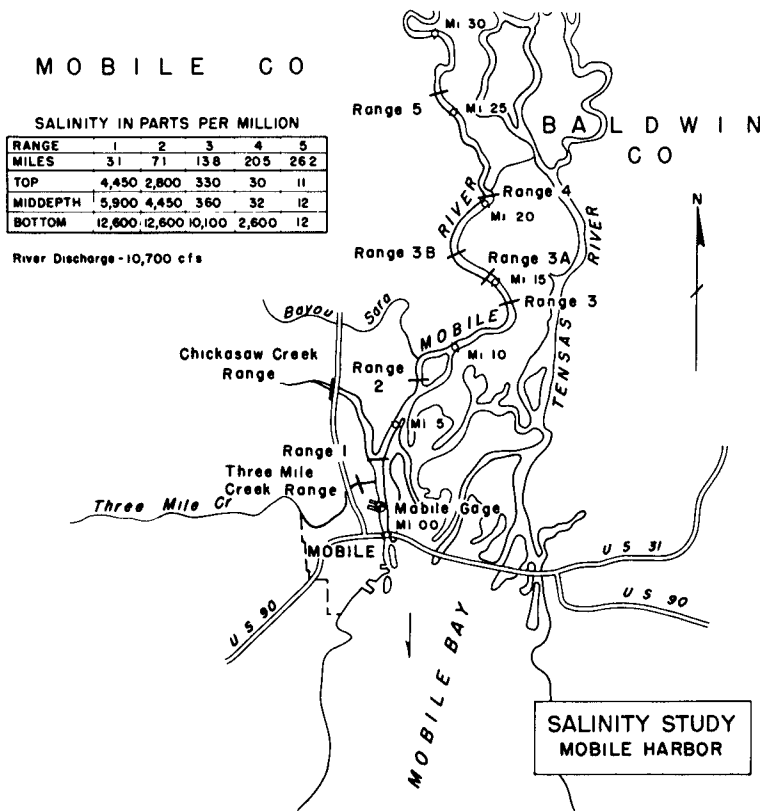
Fig. 2.

the center, compounded of the velocity around the center and the rate of
advance of the storm, blow toward the shore and do the most damage to
coastal installations; winds west of the center blow offshore and do less
damage. Maximum sustained wind velocities of record are about 84 miles an
hour at Key West, 75miles an hour at Tampa, 55 miles an hour at Apalachicola,
91 miles an hour at Pensacola, 87 miles an hour at Mobile, and 66 miles an
hour at New Orleans. Such hurricanes have often breached the barrier-island
strip, creating new inlets, some of which have closed rapidly, others remain-
ing open for many years. Hurricane attacks have also contributed to closing
of inlets, and to their migration from one location to another.

WAVE ATTACK

The direction and intensity of wave attack on the shore varies with the
winds, the trend of the shore line, and the steepness of the offshore bottom.
Between Apalachee Bay and Anclote Keys, where the offshore bottom is very
flat and rocky, storm waves break well offshore and are dissipated before
they reach the coast. From Apalachee Bay west and from Anclote Keys south,
where the mainland shore is generally protected by coastal islands, the prin-
cipal wave attack is suffered by the seaward shores of the islands, although
the bays and sounds in many places are of such expanse that regeneration of
substantial waves occurs. These attack the mainland shore. Also storm tides
and waves, particularly during hurricanes, often overtop the low barrier-
beach islands and flow across into the coastal waterways. Generally, however,
the wave attack on the mainland shore is comparatively light. The beach
islands are at times subject to heavy wave attack and even overtopping and
breaking during hurricanes. Where the islands have been improved, consider-
able damage results. Shore erosion, usually gradual, is in progress on many
of the coastal islands and at several places on the mainland shore. There
has been a considerable investment of local money in groins, sea walls, and
other protective works, most of which have met with indifferent success in
holding or building up the beaches, due chiefly to uninformed planning and to
use of inadequate materials.

LITTORAL DRIFT

The trend of littoral drift along the east Gulf coast also varies with
the wind and wave direction and intensity, and the trend of the shore. Be-
tween Apalachee Bay and Anclote Keys, where the waves are dissipated well
offshore on the flat bottom, there is practically no littoral drift close to
the shore line. As a result, that section of the coast is characterized by a
lack of sand beaches, the salt marsh extending practically to the water's
edge. West of Apalachee Bay the predominant drift is, with a few exceptions,
from east to west; at the entrance to St. Andrew Bay a local reversal is
indicated, and along the mainland shores of Mississippi littoral drift seems
to be lacking. South of Anclote Keys the predominant drift appears to be
from north to south, with exceptions such as at Little Pass at the south end
of Clearwater Beach Island where the predominant movement is from south to
north. At the entrance to Tampa Harbor, the predominant drift along the keys
north of the entrance is from north to south, whereas that south of the en-
trance is from south to north. In fact this change in direction of along
shore currents at one side of inlets and openings in the island chain is
rather common. It is probably caused by flow of the tidal currents through
such openings.

SALINITY AND TEMPERATURE

The salinity of Gulf water near the coast is in general less than the mean for sea water, and the average surface temperature is higher than along the Atlantic seaboard. The mean densities and surface temperatures of the water along the Gulf coast are typified by the following:

Place	Mean densities	Mean surface temperatures
Key West, Fla.	1.0270	79.5°
Cedar Key, Fla.	1.0194	72.4°
Pensacola, Fla.	1.0121	71.4°
Eugene Island, La.	1.0024	70.2°
Galveston, Texas	1.0170	72.8°
Port Isabel, Texas	1.0248	74.8°

It is apparent that the lowest salinity and surface temperatures are found at the mouth of the Mississippi River, and that they increase with increasing distance both east and west of that point.

DETERIORATION OF STRUCTURES

CORROSION

Because of the lower salinity of Gulf water and the less severe wave action with a correspondingly less intense abrasion by moving sand, the problem of corrosion and abrasion of steel in or exposed to Gulf water is in general not so extreme along the east Gulf coast as it is in many other locations on salt water. Furthermore, most of the steel structures subject to corrosion and abrasion are parts of harbor installations at the mouths of rivers or in indented bays, where salinity and wave action are even less than in the Gulf. Nevertheless, deterioration of steel must be expected in planning harbor structures. In recent years the tendency along the east Gulf coast has been to use creosoted timber or concrete rather than steel in coastal structures. But the development of cathodic protection has heightened the practicability of the use of steel. Some very interesting work on cathodic protection of steel structures has been done by oil companies in connection with drilling operations off the Gulf coast of Louisiana.

MARINE BORERS

The use of timber rather than steel or concrete is subject to the principal drawback that marine borers flourish in the warm waters along the Gulf coast. The most prolific species seem to be several types of Bankia, but limnoria, teredos, martesia, and sphaeroma are also found. The concentration of these organisms varies greatly from place to place and from time to time. Thus creosoted-wood bulkheads at Caseys Pass, Florida, constructed in 1937, were completely eaten away between mean low water and mean high water within 12 years, whereas a creosoted-timber retaining wall around the U. S. Engineer Reservation on Seddon Island in Tampa Harbor, built about 1921, is still in good condition after 30 years. The difference may be due to the comparative freshness of the water near the mouth of the Hillsboro River in Tampa Harbor, or to the fact that the harbor water at the engineer reservation is grossly polluted by sewage and industrial wastes, which have

been found to repel the borers. A very complete account of marine borers and their activities is contained in a publication of the National Research Council entitled "Marine Structures, their Deterioration and Preservation", by Atwood and Johnson.

Untreated timber installed along the East Gulf Coast may be expected to be destroyed in about one year. Two principal methods of combatting such attacks on timber structures have been in use along the east Gulf coast. The usual method, as used by the Corps of Engineers, has been to impregnate the timber with coal-tar creosote to an absorption of 22 pounds of creosote to each cubic foot of timber. Depending on environmental conditions, this will delay attack until the creosote is leached out of the timber, which general-ly requires about 15 years. The average life of such treated timber has been found to be 12 to 15 years, although under more favorable conditions well-treated timber has lasted as long as 20 to 30 years. The other preven-tive method has been encasing the timber piles in cast-iron, concrete, or terra cotta pipes, from below the mud line to above normal high water which affords positive protection unless the casing is broken. Even with this method it is advisable to give the pile a preservative treatment to prevent the portion above the jacket from decaying before the remainder has deteri-orated.

FEDERAL PROJECTS

The principal Federal projects along the east Gulf coast are the Gulf Intracoastal Waterway, the harbors at Port Boca Grande, Tampa, Port St. Joe, Panama City, Pensacola, Mobile, Pascagoula, and Gulfport, and the shore-protection works along the shore of Harrison County, Mississippi. Twenty-nine lesser projects have been authorized along that section of the Gulf coast. To June 30, 1950, the total Federal first cost for new work on all projects was over $23,000,000, and the total cost of maintenance nearly $20,000,000, a grand total of about $43,000,000.

THE INTRACOASTAL WATERWAY

The Gulf Intracoastal Waterway has been completed to 12 feet deep and generally 125 feet wide from Brownsville, Texas, to Carrabelle, Florida, a waterway distance of 1,073 miles. The waterway follows protected coastal sounds and connecting land cuts, and presents only the coastal-engineering features normally associated with such a waterway. Two additional sections of the waterway have been authorized but not yet provided - one from Carra-belle eastward to Apalachee Bay via Carrabelle, Crooked, and Ochlockonee Rivers, the other from Anclote River, Fla., south to the Caloosahatchee River (the latter section 9 feet deep and 100 feet wide). These also follow coastal waterways and connecting cuts, and present no unusual engineering problems.

The final link in the waterway, a channel 12 feet deep and 125 feet wide between Tampa Harbor and Apalachee Bay, is now under study. It pre-sents the unique feature that, to avoid so far as practicable the large amount and excessive cost of rock excavation entailed by an inshore aline-ment through the coastal marsh, plans are being considered to aline about 180 miles of the channel generally along the 3-foot depth contour, from 1 to 4 miles offshore in the Gulf. To compensate for the absence of

protective offshore islands, it is contemplated that the material dredged from the channel (about 53 percent rock) will be deposited offshore of the channel to create a breakwater parallel with the channel, with a crest elevation of 6 feet above mean low water, a crest width of 10 feet, and side slopes of 1 on 15. This plan may be found practicable because of the high percentage of rock in the breakwater and the usual minor nature of the waves and swells resulting from the wide expanse of shallow water along this portion of the coast. Gaps would be left in the breakwater at intervals sufficient to allow for outflow of water discharged by streams entering the Gulf in this stretch and to avoid a major dislocation of tidal and salinity conditions in the water between the breakwater and the shore. The 3-foot depth was selected for the channel because the material dredged from that depth would approximately equal that required for the breakwater. Preliminary cost estimates indicate that such a waterway would cost at least $15,000,000 or $20,000,000 less than an equivalent inshore route. There is, of course, no assurance that this interesting plan will be approved and installed.

HARBORS

The deep-water harbors along the Gulf coast are similar in their characteristics. In each case the Gulf entrance passes through an offshore bar and is shoaled by encroachment of littoral drift. Because of the moderate tidal ranges, wind and wave action, and littoral drift, entrance jetties have not been found necessary. Only one harbor (that at Panama City) has a jettied entrance, and the jetties there are only 700 feet long and spaced 1,500 feet apart. Project depths in the bar channels and in many of the inside channels have been maintained with sea-going hopper dredges at reasonable costs.

Only two streams along this section of the Gulf coast - the Apalachicola and Mobile Rivers -bring down appreciable quantities of silt into the coastal waterways. At the mouth of the Apalachicola, the river silt has built out a deltaic mud shoal into Apalachicola Bay; the silt not deposited in that shoal is carried westward by the prevailing easterly and southeasterly winds and waves, and deposited largely in St. Vincent Sound, which has maximum depths of about 5 feet. Maintenance of the project channels in Apalachicola Bay involves repeated dredging of this river-borne silt, and involves no special engineering problems.

The channels to the harbors at Mobile, Pascagoula, and Gulfport traverse the coastal waters of Mobile Bay or Mississippi Sound for considerable distances to their termini at the ports. In all cases fairly rapid shoaling occurs. These channels are maintained with hydraulic pipe-line dredges, the dredged material being deposited in the waters of the bay or sound some 1,200 to 1,500 feet from the channel. An interesting aspect of that dredging is that, despite the large amounts of material so deposited over the years, no considerable spoil banks or shoals have been created in the deposit areas; the light material is apparently picked up rapidly by waves and currents and redistributed over the bottom, probably in part back into the channels from which it was dredged.

SEAWALL, HARRISON COUNTY, MISSISSIPPI

Among the less-usual coastal-engineering problems encountered on the east Gulf coast was the study of beach erosion along the shore of Harrison County, Mississippi, between Biloxi, Miss., on the east and Henderson Point at the entrance to St. Louis Bay on the west - a distance of 27 miles. That section of shore faces on Mississippi Sound; the barrier islands and shoals separating the Sound from the open Gulf are from 8 to 12 miles offshore. U. S. Highway 90, the main artery for vehicular traffic between Jacksonville, Fla., and New Orleans, La., and points west, closely follows the mainland shore line throughout that reach. Shore property has been highly developed, and the reach is classified as an urban area by the Public Roads Administration.

Since 1893, fourteen hurricanes have struck the Gulf coast within 150 miles of Harrison County, an average of about one every four years. Only six of these were close enough and intense enough to cause major damage to the Harrison County water front. Under the impact of storm tides and waves, the shore line has been gradually receding for many years. In 1925-28, Harrison County built a sea wall along the entire 27-mile reach between Biloxi and Henderson Point, at a cost of some $3,400,000. Twenty-four miles of the sea wall is a concrete step-type structure; 1.31 miles is a concave-front type with a flanged sidewalk; and 0.65 miles is a convex-front type. The remainder consists of a small amount of masonry wall and harbor structures

When the sea wall was built, a beach from 80 to 200 feet wide was left between the wall and the water. With a few minor exceptions, that beach has disappeared, depriving the sea wall of such protection from wave attack as it originally afforded. Indications are that the beach material was moved offshore into deeper water of the sound rather than alongshore.

Disappearance of the protective sand beach exposed the concrete-sheet-pile curtain wall at the toe of the sea wall to direct wave action at all stages of tide. Corrosion destroyed reinforcing steel in numerous places where insufficient concrete cover had been provided. Settlement cracked the wall in many places. The hurricane of 1947 over-topped the sea-wall, breached the weakened wall in five places, destroyed the cut-off wall in several places, and washed out backfill, much through holes left by careless construction. U. S. Highway 90 was undermined, making it impassable at many points.

Largely because the extensive damage to U. S. Highway 90 established a considerable Federal interest in the repair and maintenance of the sea wall, a study of the requirements was made by the Corps of Engineers and a report, published in House Document No. 682, 80th Congress, 2d session, was made to Congress. As a result, the River and Harbor Act of June 30, 1948, adopted a Federal project authorizing the expenditure of $1,133,000 of Federal funds to aid in financing the repair of the sea wall and to provide a protective beach 300 feet wide at mean sea level along the entire sea wall, by pumping sand from 1,000 feet or more offshore in Mississippi Sound, on condition that local interests accomplish certain prescribed repairs of the sea wall and certain alterations in the drainage system. The work is under way, and is scheduled for completion about February 1952.

SALT WATER INVESTIGATION, MOBILE HARBOR

Another somewhat unusual coastal-engineering undertaking on the east Gulf coast was an investigation of the behavior of salt water in Mobile Harbor made by the Mobile District, Corps of Engineers, during 1945. The study was made at the request of local interests to determine the effect of salt water intrusion on the establishment along the harbor water front of industrial plants requiring fresh water. Fourteen sampling ranges were established - nine on the Mobile River between Mobile and the confluence of the Tombigbee and Alabama Rivers, one each on the Tombigbee and Alabama above their confluence, one on the Tensaw River near its source, one on Three Mile Creek, and one on Chickasaw Creek. The general features of the harbor and connecting streams and some of the more important sampling ranges are shown on Figure 2. water samples were taken about once every two weeks; in the main river, nine samples were generally taken at each station at each sampling - one each at top, mid depth, and bottom on each of three verticals spaced one near each bank and one on the center line. The 2,320 samples were analyzed in the laboratory to determine their salinity, turbidity, color, hydrogen-ion concentration, and total hardness. The results were published in House Document No. 773, 80th Congress, 2d session.

The most extreme measured conditions of salt water intrusion during the study are shown by the samples taken on November 9, 1944, with a river discharge of 10,700 second feet. The results are tabulated on Figure 2. The figures represent parts per million of salt, measured as chloride.

The study resulted in the following findings:

       a.   Salt water penetrates into the river principally along the bottom, owing to its greater density.

       b.   Tidal fluctuation has little effect in varying the extent of salt-water intrusion in Mobile River, probably due to the small tidal range and the diurnal nature of the tide.

       c.   The upper limit of salt water intrusion moves up - and downstream with decrease and increase in fresh-water discharge.

       d.   With a fresh-water discharge of 50,000 second-feet or more, the salt-water wedge is pushed back to or beyond the mouth of the river, and the entire river is fresh.

       e.   As the fresh-water discharge falls below 50,000 second-feet, salt water enters the river and pushes its way upstream. When the discharge decreases to 10,000 second-feet (about the minimum daily discharge for an average dry period), the upper limit of the salt-water intrusion is about 21 miles above the mouth of the river. In unusually dry periods, the salt-water wedge would probably penetrate somewhat farther upstream.

COASTAL PROBLEMS REQUIRING IMPROVED METHODS

The preceding study, the investigations described, and consideration of other coastal engineering problems emphasizes the need for additional information, further research, and improved methods for deriving solutions. A

relatively quick and economical method of measuring quantity of drift material, and information that will lead to an intelligent estimate of such material which will deposit in a dredged channel and will bypass such a channel are needed. At the present time dependence for obtaining these data must be placed in performance of actual structures usually situated elsewhere than the locale of the considered improvement. With this information a reasonably accurate conclusion can be drawn as to the need for jetties, and maintenance costs which affect, sometimes strongly, the economic practicability of a project can be correctly appraised rather than imagined. Moreover should it be possible to measure the quantity which moves in various strips along the bottom, the correct or economic length of jetties could be foretold. With an indication of the direction of movement as well as the quantity, a conclusion can be drawn as to whether or not one jetty will suffice rather than two.

The relatively short life of structures erected in sea water has been pointed out. Research should continue on means of preventing marine borer attacks, on methods of protection against corrosion, and on the development of low cost non-corrosive metals and alloys.

The procedure for determining salinity, turbidity, etc., is cumbersome and costly. Methods of simplification are badly needed to facilitate solution of many of our harbor problems.

Model studies should be used to a greater extent than in the past. At the present time their cost often prohibits their use in connection with small projects, but even if comprehensive tests cannot be justified, perhaps partial tests can furnish beneficial results. Model tests can show where dredge spoil should be placed to minimize its return to excavated channels. Without the use of models only the costly cut-and-try process is available, though this may be guided by a float study. This problem in general has not been attacked in a scientific manner.

The model may be able to give an indication of the value of overdepth dredging for advance maintenance purposes as well as the relative rate of shoaling to be expected when a channel is deepened. At present the engineer must wait until the job is done and then the conclusions are often masked by other changes which have been made.

Excellent information on salt water intrusion can be obtained from models. When there have been a large number of instances of verification with few or no failures, the engineer may develop sufficient confidence to depend upon the model for these results rather than to employ the costly method of sampling and chemical analysis.

It has been shown at times in the past that orientation of channels through bars affects the maintenance of the channels. The bar channels of the Mississippi River are outstanding examples of this. But a statement of principles and a method of analysis which will indicate expected shoaling on various alignments and relative permanence of those alignments are needed. In the absence of means of analysis, the model can be used, it is believed, to furnish qualitative answers.

The model can also show waves and surges to be expected in bays and harbors with various configurations and arrangements of inlets and inlet works as well as structures, needed to remedy unsatisfactory conditions. By permitting trials of various jetty designs and alignments it can reveal the plan most likely to require the least channel maintenance.

Though models can be used advantageously to a greater extent than in the past, they are not a cure-all. The results, at times, can be misleading. They should never be accepted without critical scrutiny. Instead all model tests should be carefully analyzed by engineers of appropriate training and experience. Each observed action should be explained so that the engineer is satisfied that corresponding causative influences exist in the prototype and similar effects may be expected. By close observation and correlation with hydraulic laws and principles, analytical methods may be developed which will permit dependable forecasting of many results without the use of models. This should be the aim, for with its consummation, the cost of arriving at correct problem solutions will be reduced, and coastal engineering will be placed on a rational rather than an empirical basis.

REFERENCES

Doremus, E. P., and Doremus, G. L. Cathodic Protection of Fourteen Off-
shore Drilling Platforms: Corrosion, Vol. 6, No. 7, National
Association of Corrosion Engineers.

Atwood, W. G., and Johnson, A. A. Marine Structures; Their Deterioration
and Preservation: National Research Council, Washington, D. C.

Larrabee, C. P. Corrosion of a Steel Pile During Twelve Years of Service
at Pensacola, Florida: Carnegie-Illinois Steel Corporation.

Reports of the Corps of Engineers, U. S. Army

Chapter 21

ECONOMICS OF COASTAL STRUCTURES

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The engineer is constantly called upon to make an economic analysis of proposed engineering works. His analysis is just as trustworthy as his base information. In some cases experience and knowledge are so complete that an estimate can be accepted with assurance of great accuracy. The purpose of this paper is to discuss some of the factors involved in making an economic analysis of coastal structures.

The coast line of this country provides many examples of all types of structures in various degrees of preservation and effectiveness. Some of those in the best physical condition after a long period of time show little evidence of past effectiveness. At the other extreme are structures in poor physical condition and obviously ineffective. To determine whether structures have been both functionally effective in their life time and of sound construction to withstand the elements to which exposed would require very careful analysis, but such an analysis would give a measure of the expense of providing an engineering solution to the problem involved. It would provide an answer to the question of whether it was worth the cost. The designing engineer is not always in a position to make such a post mortem case study, his principal attention being given to making estimates of the future. However works of governmental units investing in such works provide material for constant appraisal of results which can be used to bolster estimates for future work. Governmental units, as private owners, are concerned with getting the maximum returns from investments. Engineers serving such masters must take cognizance of past experience in order to improve service in the future.

In utilizing works of the past, or in designing current structures, several elements must first be evaluated. These may be listed as follows:

a. What is the purpose of the structure? What is its value to the owner, assuming functional adequacy of structure? This is expressed in terms of the benefit in Corps of Engineers procedures. Obviously the value of benefits, acceptable to the owner, must exceed the cost of the structure.

b. Was or is the structure functionally adequate for the purpose? This question, insofar as coastal structures are concerned, is more easily answered for the past than stated with assurance for the future. A structure which has been in use for several years will show by its action whether it has accomplished all that had been anticipated, with minimum disruptive collateral effects. A functionally adequate structure is one that accomplishes its purpose rapidly, constantly, and with minimum additional works. It will be that structure which is located and layed out

with reference to all known forces so that it accomplishes the purpose
with the most economical structural design requirements.

        c. Was or is the structural design adequate for the exposure?
Again, judgment must be used in evaluating this factor. One can be
charged with expensive over design as well as underdesign. Was there a
calculated risk involved and was it properly evaluated?

These questions will be discussed with particular reference to studies of the Beach Erosion Board.

EVALUATION OF BENEFITS

An engineering study is undertaken when local authorities believe
that an engineering solution to their problem may be justified. This pre-
supposes that the anticipated benefits would justify the expenditure of
funds. While some of the benefits may have to be reevaluated and the
suggested engineer solution may open up additional benefits the basic
estimate of benefits precedes the engineer study.

Beach Erosion Board studies are concerned with shore protection and
collateral effects on the shore line caused by navigation structures.
Benefits are therefore associated with preservation of a shore line.
Since a beach is often involved, special consideration of benefits of a
beach must be included in appropriate cases.

Benefits under these circumstances are classified as direct and in-
direct. The direct benefits are those which can be evaluated with some
accuracy whereas the indirect are intangible and only related to an inde-
terminate degree with the engineering improvement. Direct benefits are
further broken down into damage prevented, enhanced value of shore proper-
ty by reason of the improvement, and, in certain cases, recreation.

The usual manner of calculating damage to shore property is to consi-
der the value of land lost annually, due to wave action based on the value
per square foot. This method assumes an unlimited area of constant value
which can be eroded. In developed shore property this is not the case.
A lot may be of a certain value when 100 feet deep but quite a different
value per square foot after the seaward 75 feet have been eroded. Un-
checked erosion may therefore introduce a further direct damage due to
changed use which can also be expressed as an average annual damage. In
like manner roadways and utilities may introduce special types of direct
damages due to unchecked erosion. An air strip, if shortened or breached
unduly, could destroy the effective use of an airfield. Thus direct
damages prevented must be given individual treatment to fit each case and
the amount reduced to an annual rate to facilitate comparison with other
costs.

Upon the assumption that erosion can be checked, property values are
enhanced to the extent that higher uses are then opened. Property values
without improvements provided later by new investments are involved. With

an assurance that further loss of land or structures will not be caused by encroachment of the sea, property may be used for more permanent or higher class uses. In the case of roads and utilities it may justify the location of more important structures along the shore line. In all these cases, enhanced value of the existing land or property only is considered. This is expressed on an annual basis by a fair rate of interest on the increase in valuation.

While a beach is classified as a protective structure it also has value as a recreational asset in most cases. In beach resort communities such as Atlantic City it represents such a valuable asset that expenditure of large sums of money can be justified in preservation of the recreational beach. The benefits of a beach must therefore be evaluated consistent with the local situation. This varies from being the central asset in the economics of the community, through a simple community recreational beach park to a beach having current value solely as a protection to the shore line. Where the beach has a value there may be justification for holding or restoring it to its optimum width for the purpose intended, whether for recreation or protection of the shore line. Benefits of stabilization of the beach width may be calculated then as for the shore property in terms of damages prevented and enhanced value per square foot of beach. Such evaluation considers the beach to have a value as property.

In determining the value of a beach for recreation consideration is further required of several factors contributing to an optimum beach such as:

a. The material in the beach. Sand of median diameter less than .4 mm. is preferable.

b. The slope of the beach. About 1/50 is optimum with a berm width between 100 and 200 ft.

c. The suitability of the water. It must meet public health standards to be used as a beach and the temperature for bathing should be between 65° and 80° F.

d. The accessibility to users. Communications must be adequate and population within using distance should be sufficient to support a beach population of one person to each 100 sq. ft. of beach above H.W.

e. The amenities provided. Bathhouses and pavilions should be adequate for the designed beach population.

The popularity and value of a beach must be judged by its current or past use, or by analogy in relation to other beaches of the same character. Because of the many intangible factors involved, judgment must be formed

of the probable beach population which would use the beach, regardless of how provided, and what the value would be to those using it. The factors can be evaluated by a careful survey of public opinion using recognized statistical procedures, but as a workable average, 25 cents per person per visit using the beach up to a population of one person per 100 sq. ft. of beach may be used and beyond that a flat value of 25 cents per day of use per 100 square feet of beach above high water. These estimates when adjusted for cost of operation and maintenance of a beach for recreation will permit calculation of the annual recreational benefit of a given beach. Deviations from the average should be supported by careful analysis of the factors contributing to the optimum beach.

A resort community, such as Atlantic City, presents a special problem in that the business life of the community depends upon the fame and popularity of the beach, even though relatively few visitors to the community go there for bathing, but rather to view and mingle with the bathers. The beach may not be the sole reason for the intensive business conducted in the community but the business life would wither without the beach and therefore as in Atlantic City, business will support any beach expense within reason regardless of number of bathers. A measure of the importance of a beach to business will be found in the difference in the amount of business during the bathing season and the non bathing season. Not all of this difference can be attributed to a good beach but a certain proportion may be so credited as a benefit. Depending upon the importance of the beach to the resort business, from 10 to 25% may be taken. Not all people will agree on this allocation and these figures are offered only for consideration. The effect is to place a value on the beach well beyond that which would be found in a beach park. Because of the difficulty in assessing the value of business attributable to the beach the Beach Erosion Board has favored considering this an indirect benefit.

Other indirect benefits are found in benefits to the community at large such as reduced juvenile delinquency or crime, and a more healthy and better adjusted citizenry.

DETERMINATION OF FUNCTIONAL DESIGN

A proper functional design depends upon a correct appraisal of all the forces involved. Since, on a shore front, these forces are rather complex, with daily and seasonal variations, and occasional storms of greater intensity, the best functional design must be that which meets the most general condition with a special provision for intensive storms of short duration. Even general conditions may be slowly changing with long time trends, and one section of a shore amply protected by a beach for example, may find in a term of years that accreting conditions change to erosion. Surveys over a period of years show shore line and off shore changes in areas not affected by works of man. A major portion of the Beach Erosion Board research is directed toward the analysis of forces affecting shore processes so that the relation between cause and effect will be more

clearly understood and considered in functional design.

Beach erosion control structures must consider the movement of beach materials in littoral drift. Changes in distribution and rates of littoral drift are found even in short reaches of a beach. Evidences of these changes are noted in changing beach profile, in minor changes in beach materials, and the action of groins, which are more successful in one spot than in another. These differences are caused by irregular distribution of forces or materials and emphasize the need for a complete understanding of the forces before applying a remedy. It is evident that a single solution cannot apply to long reaches of a coast even though the average rate of littoral drift may be known. The Beach Erosion Board is investigating the affects of bottom irregularities and currents upon approaching wave trains in an effort to establish a basis for the irregular distribution of energy.

The direction of littoral drift changes along some sections of the coast due to changes in direction of approach of waves at different seasons of the year, or to affects of occasional storms. This results in building of beaches under certain conditions and erosion under others. The type of waves, whether long or short period, and low or high, also has a varying effect on the shape of a beach.

A better understanding of the manner of application of energy and the resultant action of shore materials under different conditions and over a long period of time will improve the design of methods or structures to correct unfavorable conditions. It will help settle the question, in an eroding shore, whether artificial fill, or groins of some type, length, and spacing, would give the best and most economical protection. It will indicate where jetties can be used to advantage and where they would have little effect or would cause more harm than good. It would give greater assurance that the location and type of structure would keep maintenance costs in connection with the improvement to a minimum. In short better functional design demands a more accurate definition of the applied energy and resultant action of beach materials. Increased knowledge of these factors would save much that is now being spent in measures of desperation, or in expensive improvements which ignore, for example, cyclical rises in lake levels, or points on the coast having unusual concentration of wave energy.

STRUCTURAL DESIGN FACTORS

Structural design follows correct functional design. After a careful analysis of all the functional factors develops the proper location, type, and general plan of a corrective method or structure, there remains the important problem of selecting materials and designing a structure that will withstand the forces and resist the corrosion and erosion due to exposure for a reasonable life expectancy, all things considered. It must be assumed that the most reasonable annual costs are desired.

Expressed in money this means that the annual interest on the investment, plus amortization charges, plus annual maintenance results in the lowest annual charges.

The amount to be invested in a coastal structure should consider the probable period in which there will be no material change in the functional requirements. A study of historical changes may give a clue to the answer to this factor. Obviously there would be no point in building a structure that would either have to be moved after a period of years, or left to deteriorate without any further functional need. Some structures, such as groins, may, if properly designed and located functionally, work themselves out of any further functional use in a relatively short time. The materials chosen should be suitable for the period of probable need for the structure. The more durable, and hence the more costly, may be chosen for those elements requiring the longest life.

Having determined the desired useful life of a structure based on functional need, structural design must be based on the expected exposure and on the day to day forces acting upon the structure. Consideration must also be given to reversal of forces and to storm forces. The maximum stresses are probably developed during storms and therefore an analyses of storms in the area is necessary. A structure probably should be designed for a storm expected to occur at least once during the useful lifetime of the structure, but judgment of the affects upon related facilities will probably be required before selecting less frequent but more severe storms as a criterion. In some structures, as timber groins, it may not be economical to protect a whole field of groins against a contingency which may affect only one or two groins in minor degree. For this reason it is not reasonable to put expensive rip rap around the outer end of all timber groins because an unusual storm destroys the outer end of one groin. In the end, the assumed design stresses must be considered together with the durability of the materials in determining the proper type of structure.

The life of materials in structures along the coast depends upon the climate and the exposure. Local experience probably is the best guide, though the Beach Erosion Board is undertaking to tabulate and analyze data concerning this. The task is complicated by the fact that even in the same climate, the exposure of similar types of structures and materials may be quite different.

ANNUAL CHARGES

With or without adequate experience, the ultimate cost of a structure, as expressed in annual charges, depends upon the validity of the assumed useful life of the structure. This may vary for coastal structures, from a few years to 50 years. The Beach Erosion Board policy, in conformance with the policy of the Corps of Engineers does not extend the financing of any structure beyond an assumed life of 50 years. Even where there may be a functional need for 50 years, it may be necessary to consider replacement of the structure or elements of it, one or more times during that

period, and the annual charges reflect that condition.

The assumed useful life of a structure affects the amortization rate directly. It therefore has an important bearing on the estimated annual charges and should be realistic if a comparison with annual benefits is to have any value. When replacements will be required during the useful life, an amount must be set up initially which will represent the present worth of the estimated future cost of the replacement. This, added to the initial cost, must then be amortized over the useful life of the structure.

The third element in computing annual charges after interest on investment and amortization, is maintenance. The cost of maintenance will depend upon the materials used in construction, and upon the exposure. Again, local experience is probably the best guide, because there will be many factors to consider, most of which are peculiar to the locality. Aside from the physical needs for maintenance, the cost will, in general, vary over the life of the structure in about the same degree as the value of benefits. Since this will be largely unpredictable, though at present rising at a rapid rate, it would probably be better to keep all estimates on the basis of present values.

Maintenance costs will be higher for meagerly designed structures than for excessively designed structures. The proper balance between initial cost and maintenance is therefore one that requires judgment. It may be desirable to consider several alternate designs in order to develop the best combination for total low annual charges. The result will give the annual charges combining the best functional design with the most economical means of carrying it out.

BENEFIT COST RATIO

The relation between the annual benefit value, and the annual charges for the improvement, gives the benefit-cost ratio, which, if substantially greater than unity, shows economic justification for the improvement. This does not necessarily mean that the improvement should be built immediately, for the owner must weigh the proposed improvement against his financial capability and the relative merits of other prospective improvements. This latter determination is usually beyond the scope of the engineer.

Chapter 22

REDUCTION OF MAINTENANCE BY PROPER

ORIENTATION OF SHIP CHANNELS THROUGH TIDAL INLETS[1]

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ABSTRACT

The inherited courses of some bay-port ship channels take them
through tidal inlets along courses running across dominant directions of
strong current movement and scour, or on courses that upset the natural
tidal regimes. Such discordance may make necessary excessive maintenance
dredging. Geological study of a section of the Texas coast shows that,
in a unit coastal environment, there may be a predictable stable position
of a tidal inlet and a common stable orientation for its channel which
might better have been used for the ship channel outlet. Among probable
damages to the natural environment resulting from a wrong orientation is
excessive sedimentation in the inlet channel.

Engineering studies are needed to determine the economics of re-
orientation and relocation of misfit channels of the type described.

The tidal inlet or "pass" is the central channel of a tidal delta.
The delta is an enlargement of a barrier sand island at a gap where tidal
and other flow into and out of large inland water bodies forms a strong
local field of force with a longshore sediment drift and current.
Engineering works in this field of force should utilize its character-
istics, not fight them. As the coastal section studied here is only one
of many, extension of the geologic study, with accompanying engineering
studies, should be made to permit general laws of inlets, tidal deltas
and barrier islands to be set up for both geology and engineering.

INTRODUCTION

THE TIDAL INLET OR "PASS"

The oceanic entrances to most harbors where a barrier island[2] of
sand is encountered a few miles offshore are usually located at natural
tidal inlets or "passes". In the natural state, such an inlet has a deep
natural channel terminated at each end by a so-called "bar", Fig. 1. Many
tidal inlets migrate.

Geological studies on the Texas coast suggest remedies for some of
the difficulties in maintaining ship channels through inlets. Such
studies might be used as guides in channel location and orientation on
other coasts where similar conditions are found to exist.

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[1]The Agricultural and Mechanical College of Texas, Department of
Oceanography, College Station, Texas. Contribution No. 12.

[2]Synonym: Barrier beach. Improperly: "Offshore Bar".

ACCIDENTAL CHOICE OF LOCATIONS OF "IMPROVED" SHIP CHANNELS

Historically, most improved ship channels have followed in general the passageways used by early shallow-draft shipping, being tied down to installations of the earlier period. When "improvements" of the channels were attempted, the locations of channels and harbors and the engineer's own choices of "best" routes were in some cases such as to bring him and his works into conflict with the existing equilibria developed between exceedingly strong natural forces.

STABILIZATION AND MAINTENANCE OF INLETS

To prevent migration of inlets used for ship channels, engineers have built retaining walls or revetments along one or both sides. To remove the bars, frequent dredging has to be carried on, as the "bars" tend to re-form after removal. Jetties help but cause beach erosion.

REDUCTION OF ENGINEERING COSTS BY
APPLICATION OF GEOLOGIC PRINCIPLES

RELOCATION OR REORIENTATION

Artificial stabilization and continued maintenance of some ship channels through tidal inlets is very costly. It is possible that some such excessive maintenance operations are due to (1) wrong location of channel, (2) wrong orientation of pass through barrier island. Where one or both of these conditions seems to occur - for example, the Corpus Christi ship channel and Aransas Pass, Texas - combined engineering, hydrologic and geological studies are needed to determine whether there is a remedy. The questions arise: (1) if the pass were reoriented, or (2) if the channel were relocated, would there be a great enough saving to amortize the cost of the change? Some maintenance costs are shown in Table 1.

ASSISTANCE FROM GEOLOGY

On some coasts, geologic study shows (1) that stable channel locations can be found where the tendency of the inlet to migrate is absent, and (2) that, regardless of location, passes can be oriented, or re-oriented, so as to permit maximum natural flow through them, with presumed reduction of sedimentation in the channel. Reduction of the tendency of inlets to migrate and full utilization of natural currents should greatly reduce the costs of both original construction and maintenance. It would be the task of the engineer to determine whether the routes necessary to use these physically preferable locations for access to specific harbor sites would be economically desirable.

TIDAL DELTA - KEY TO GEOLOGICAL SOLUTION OF PROBLEM

It is evident from the constant movement of large masses of sediment along the courses of tidal inlets and their accumulation behind jetties that the inlet lies in a strong field of force. What is this field of force and what are its directions and magnitudes? Knowing these, we should be able to attack specific problems in the field with some confidence of success.

TABLE 1

SOME SHIP-CHANNEL DREDGING COSTS - TEXAS COAST

Place	Millions Yds.	$	Years	Miles	Yards/Year
HOUSTON	119	13.0	25	51	4,750,000
CORPUS CHRISTI	32	4.5	16	21	2,000,000

	$/Year		Mile/Year	
HOUSTON	520,000	87,000 yds.	$12,000	
CORPUS CHRISTI	275,000	100,000 yds.	$13,000	

Figures showing what portions of total yardage and costs figures apply to maintenance of channels through tidal deltas alone are not available. Although the above figures suggest that the total maintenance is proportional to the mileage and is not greatly excessive for any one section over the average, yet we do not know how far along a channel excessive maintenance may be caused by difficulties originating in the tidal delta. The Houston and Corpus Christi channels are dissimilar in many respects and their maintenance costs may not be comparable for the purposes of this study.

The geologist recognizes the tidal inlet and its surrounding elevations, tidal flats and submerged contour irregularities as a tidal delta, Figs. 1, 2.

STUDY OF GROSS DYNAMICS OF TIDAL DELTA

The writer is not informed that a thorough going quantitative geophysical study of the hydrodynamic field of force of a tidal delta has ever been made. However, he has accomplished a geological study of the dynamics of the tidal delta from the study of a group of inlets and deltas. This has been done by studying the regional geologic and hydrodynamic setting and determining the origins, growth modifications, migrations and conditions of stability of the inlets and deltas, as well as the geological results of the attempts of engineers to control them. This study was made on the coast of Texas in the area shown in Fig. 3.

ANALYSIS OF TIDAL DELTA AND INLET

FORM OF DELTA

The simplest form of tidal delta would be symmetrical with respect to its inlet and centrally located opposite a large bay with a large volume of inflowing land runoff. Actually, most tidal deltas are highly asymmetrical, Figs. 2, 3, with a diagonal channel, offset oceanic shoreline and unequal lagoonward lobes. There seem to be no good examples of simple symmetrical tidal deltas on the coasts of the Atlantic or Gulf of Mexico. Asymmetry of a tidal delta seems to be due to the presence of a longshore drift. Hence we draw our typical tidal delta somewhat asymmetrical, Fig. 1. Galveston Bay would be a fair example. Pass Cavallo, Fig. 2, is representative of the section of coast here studied, Fig. 3.

The scarcity of symmetrical tidal deltas on the east coast of the United States indicates, as will be shown, that a longshore sediment drift is commonly present and is stronger in one direction than in the reverse.

All tidal deltas are double, formed by the deposition of sediment during both the inflow and outflow of tidal and other waters, Figs. 1, 2. The higher waves and stronger longshore currents at the oceanic end of the inlet cause the outer part of the delta to be broadly arcuate and only slightly protuberant, but the quiet waters of the lagoon and bay allow a large lobe to develop there on at least one side of the inlet.

Tidal inlets are normally centrally deep, Figs. 1, 3, but shallow at each end. The channel may divide around a central bar or island. The inner bar may be large and crescentic, Fig. 2. The bar-like front of the delta at the ocean may contain a very large yardage of sediment and appear merely as a broad shoals.

STABLE AND MIGRATING INLETS AND DELTAS

Tidal inlets and tidal deltas are either (1) migrating or (2) stable. From what we know historically on the coast of Texas and Louisiana, tidal inlets are occasionally opened during the high waters and great waves of hurricanes in situations where they are not stable. The new inlet may
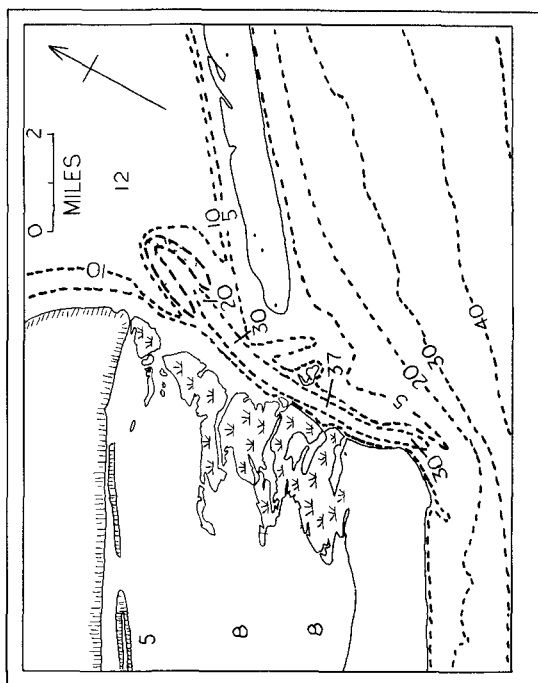
Fig. 2. Pass Cavallo and its tidal delta, Matagorda Bay, Texas.

A natural, asymmetrical tidal delta of Central Texas coast in the stable position. Crescentic channel bar across inner end, broad delta front across outer end. Inlet oriented NNE-SSW. Serrated islands of west half of delta show dominant drift in coastal lagoon. Barrier island strongly offset gulfward at down-drift side of inlet. Depths in feet below mean low water.
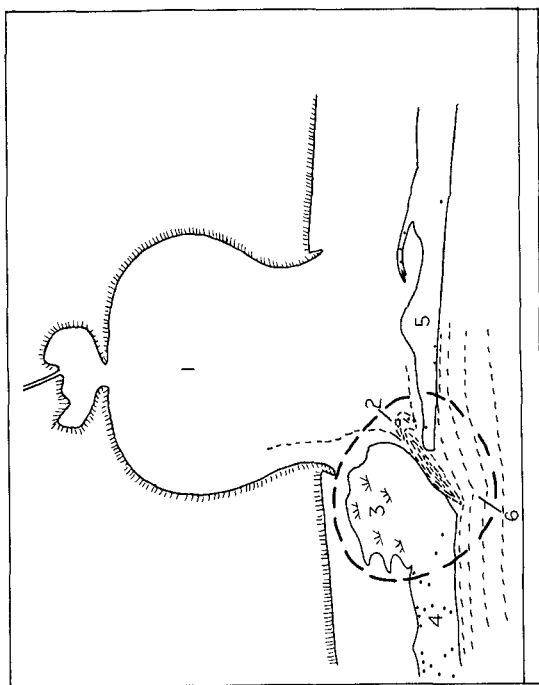
Fig. 1. A typical asymmetrical tidal delta.

Delta outlined by heavy broken line.
1. Bay with river entering inner end.
2. Inlet channel blocked at both ends by bars and shoals.
3. Down-drift lobe of delta, an expansion of barrier island.
4. Barrier island.
5. Narrow, pointed barrier island section on up-drift side of inlet.
6. Oceanic part of delta largely submerged, poorly developed.

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promptly sand up, or, much more rarely, it may establish itself with a typical deep central channel. The opening of a new inlet in a storm is probably the usual beginning of a period of inlet migration.

If not interfered with artificially or by the opening of further storm inlets, the newly established inlet will migrate slowly to a position of stability. Since the entire huge tidal delta moves with the inlet as a part of a single structure, the stable position of an inlet is marked by a heavy accumulation of sediment in the stable area. When a new inlet is formed by a storm at another point, the building of a new tidal delta begins, the former delta remaining stranded and its inlet slowly closing or becoming a shallow bayou.

DETERMINATION OF STABLE ORIENTATION AND POSITION*

Geologists can, by regional geological and meteorological studies, supported by adequate cartographic and historical data or at times in the absence of such data, determine the stable position of the tidal delta and, what is equally important, the stable orientation of the inlet channel in the delta. We will use the central coast of Texas as an example, for here the case is simple and clear-cut.

WIND DISTRIBUTION IN RELATION TO CONFIGURATION OF INNER AND OUTER SHORELINES

The wind regime of the coast of Texas is monsoonal, with the winds sub-equally divided in velocity and duration between the strong offshore and longshore northerlies (NW, N, NE).of winter and the equally strong and highly persistent onshore easterly and southerly winds (E, SE, S) of the warm and hot months. Only about 2 percent of the wind comes from westerly directions (W, SW). The heavy air of winter adds to the effectiveness of the northerly winds, nearly all of which are stormy and associated with the passage of a cold front. In our study, the longshore winds and wind components produce a strong longshore sediment drift in the coastal lagoon and along the Gulf shoreline. The offshore winds cause strong currents to course through tidal inlets - because of a funnel-shaped shoreline arrangement. Onshore winds check outgoing tidal waters and promote deposition of sediment at the Gulf end of an inlet.

DELTA MADE ASYMMETRICAL AND INLET CAUSED TO MIGRATE BY LONGSHORE SEDIMENT DRIFT

The coast of Texas southwest of the Brazos River mouth shows only asymmetrical tidal deltas, Fig. 3. The barrier island is narrow, low, bare and southwestwardly pointed on the eastern, up-current side of the inlet, but broad, lobate to triangular on the down-current side. The down-current side is always broad, may be built up high by dunes, and is commonly well stabilized by vegetation. The part protruding into the lagoon is usually low - less than 4-feet above sea - and largely marshy, grading off to broad flats and shallows that may essentially close the coastal lagoon.

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*The long-term average stability of axial position, here observed, may not prevent appreciable shifting of sediment in the inlet and at its ends.
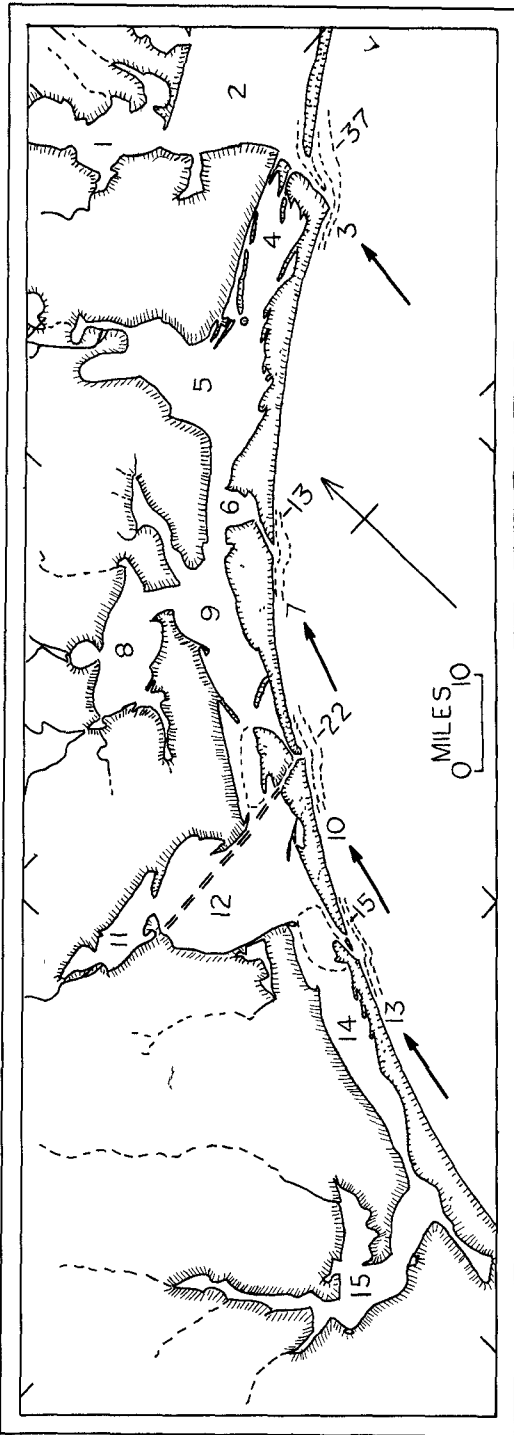
Fig. 3. Bays of central Texas coast with their natural tidal inlets and tidal deltas.

1, 2, Lavaca and Matagorda Bays. 3, Pass Cavallo (stable).
4, 5, 6, Espiritu Santo, San Antonio and Mesquite Bays.
7, Cedar Bayou Inlet (washover fans interfere).
8, 9, Copano and Aransas Bays. 10, Aransas Pass.
11, 12, Nueces and Corpus Christi Bays. 13, Former Corpus
   Christi Pass (slightly S of most stable position).
   The Corpus Christi Ship Channel now connects
   11 and 12 with 10.
14, 15, Laguna Madre and Baffin Bay, with no tidal inlet.
Original depths in natural inlet channels: -37, -13 feet, etc.

From Galveston Bay to Corpus Christi Bay records[3] show that the direction of migration of inlets before any were artificially stabilized was to the southwest. A study of F. C. Bullard (1942) of the appearance at the Gulf beach of the diagnostic minerals of the sands of the rivers of Texas confirms the evidence from inlet migration that the dominant long-shore sediment drift is to the southwest.[4] Study of spits in the coastal lagoon shows the same drift direction there, Fig. 2.

It is evident that the longshore drift from the northwest carries the sediment into the inlet, without any possibility of storing materials at the near edge of the inlet. The sand merely moves along the Gulf shore to the inlet, where it becomes involved in the back-and-forth tidal movement. Sand also drifts across the inlet to the southwest. The southwest part of the delta grows, becoming excessively large, in the stable position. A condition that helped to make the northeastern part of each delta here thin and pointed is the diagonal, NNE-SSW natural orientation of the tidal inlet through a barrier island that trends northeast-southwest.

INLET LOCATION AND NNE-SSW ORIENTATION

The southerly locations and nearly north-south orientations of natural inlet channels on this coast, Fig. 3, are due to the repetition at each large, wide mouthed bay of a combination of conditions produced by (1) the occurrence of these bays north of the barrier island, (2) the presence of strong northerly winds which funnel water over these long fetches southward through inlets, (3) the direction of inlet migration, (4) the bearing of the barrier island, (5) the presence of a strong long-shore sediment drift. Inlet migration to the southwest in conformity with the dominant longshore drift carries the delta and inlet south of the associated bay. "Northers" are dominant here during the winter monsoon. A funnel shape is provided by the angle between the southwest side of a northwest-elongated bay, the southwest-trending barrier island and its transverse inlet, Figs. 1, 2, 3. The great southward surge of the waters of the norther scours out the channel on a north-south or NNE-SSW axis con-formable to the axis of the natural funnel just described. In some cases the southwardly and southwestwardly directed forces are so strong that the delta and inlet are driven down the coastal lagoon to an extreme, somewhat unstable position, as Cedar Bayou, Fig. 3.[5]

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[3]Galveston District Office, U. S. Engineers.

[4]This observation may not apply to local building of beaches behind jetties. The longshore drift may have its seasonal reversals and has a seeming local reversal on the northwest-trending north half of Rio Grande delta.

[5]This inlet has been obstructed by a strong development of washover fans.

On this particular coastal sector, the natural north-south orientation
of inlets introduces an engineering conflict as it is diagonal to the trend
of the barrier island and not on the shortest line across it. Yet the
diagonal course is the equilibrium position in the deltaic field of force.

The strength and action of the heavily sediment-laden waters driven
southward by the norther and their passage in a flood through the inlet
has been studied and observed from the air by both the Galveston District
of the U. S. Engineers and by the writer. A "norther" is the stormy
passage of a cold air front over the prairies and the tidewater zone. For
several days, as the front approaches the region, warm winds blow toward it
from the south, roiling the bay waters, loading them with sediment and
raising the level of the bays by water blown in from the adjacent coastal
lagoon and by strong winds retaining part of the tidal prism of the bay.

With the arrival of the cold front, the wind is reversed and blows
violently from northerly directions. Additional sediment is then picked
up from banks and bottom. From the air, muddy water can be seen streaming
southward from every shoreline projection and angle. The large prism
of water previously accumulated in the bay by the southerly winds is now
moved rapidly out through the funnel of the inlet by the northerly winds.
Every year, drownings occur when fishermen in light boats are overturned by
the racing floods that go through the passes. It is the strong movement
of sediment out of the bays and southward along the coast with the northers
that makes the southwestward longshore drift dominant in the yearly drifts
and current movements. In the warm months, the strong, steady, southerly
summer monsoon is unable to affect the inlets appreciably, because there
is no funnelling action possible from the Gulf.

We conclude that meteorological factors combine with topographic
factors in the coast of Figure 3, to orient natural inlets NNE-SSW and to
move the inlets to positions south to southwest of the larger bays. These
relationships are produced because the floods passing through the inlets
from the north are adequate to orient an inlet channel along the axis
of the natural funnel occurring at the position of inlet stability.

ONE INLET TO A BAY

Other features in delta and inlet histories need to be outlined to
complete the support for assertions and interpretations made. Three lines
of evidence show that only one inlet at a time survives for more than a
short time for each bay: (1) Only one per bay now exists or has persisted
for any Texas bay for an appreciable period. (2) During hurricane floods,
scores of high-level washover channels form across the barrier islands, but
do not become inlets except in rare cases. For example, former District
Highway Engineer T. S. Bailey counted over 40 channels still discharging
lagoon waters along Padre Island the day after a hurricane of the 1930s
while the flood was still running off. The writer has studied these wash-
over channels on the ground and in air photographs made before and after
the 1930 hurricanes. These channels exist and have long existed as gaps in
the dune wall of the island and have sanded up promptly at the beach level
after each washover flood, the beach ridge being built across their out-
lets. Although all these channels were active in the 1930-1940 hurricanes
and in others since, none led to the formation of an inlet. (3) Throwing

the Corpus Christi ship channel into the natural inlet for Aransas Bay in 1924 led to the slow silting up of Corpus Christi Pass, in spite of occasional reopenings during hurricanes and by dredging. (4) Although artificial cuts were made through some of the washover channels by the Texas Game, Fish and Oyster Commission all quickly sanded up except the reopened natural inlet of Cedar Bayou.

The experience in Corpus Christi Bay showed that the tidal and other forces that keep the inlets active will support only one inlet at a time in a bay. Although there may for awhile be two adjacent openings, this is usually due to some temporary condition. Eventually one inlet will be established as deep and typical with a well-developed tidal delta, while the other will sand up and be closed. The wholly artificial Corpus Christi ship channel runs at right angles to both the tidal flow and the monsoonal winds. Moreover, this orientation gives the natural Aransas Pass inlet a dog-leg and has prevented it from reaching the position of stability which it had almost attained in 1887 when it was fixed by a retaining wall.

A CLOSED BAY SYSTEM

One of the persistent efforts of the Game, Fish and Oyster Commission to start a new tidal inlet was at Murdock Landing in the coastal lagoon southeast of the narrow opening of Baffin Bay. This bay is fed only by small creeks and intermittent arroyos and its narrow mouth is oriented so as to prevent the wind-driven waters of northers from impinging against the barrier island. Baffin Bay has not maintained a natural inlet during the century of map-making on this coast and it is evident from the topography that it has not done so for many centuries. It has two large wash-over channels, one being known as Boggy Slough. These have been shallowly open for short periods after hurricane floods or other excessively high tides.

STABLE POSITION OF INLET SSW OF WIDE-MOUTHED BAY

A single item remains to be established before we are ready to discuss reorientation and relocation again - the stable position of a tidal inlet and delta. Texas bays that have inlets are wide-mouthed and the inlets lie to the south or southwest of large bays. Inlets that seem to have been stable for centuries, judging from the large sizes of their deltas and the heavy build-up of dunes or spits on the south halves, are Cedar Bayou, Pass Cavallo and, before engineering interference, Corpus Christi Pass. Each of these inlets is located at an extreme southerly position with regard to the associated bay or suite of bays. Pass Cavallo, Fig. 2, has been stable since the recognizable accurate mapping by Cardenas in 1689, or 260 years (W. A. Price 1947, Fig. 2b).

Passes with histories of migration - all to the southwest - include Galveston Entrance, San Louis Pass and Aransas Pass. Deltaic changes have favored Brazos Santiago over Boca Chica at the southern end of the Texas lagoons.

MIGRATION PAST THE STABLE POSITION

Examination of the most southerly position taken here by natural

inlets suggests that some of them may have been driven past the most stable position and have come again into a position of instability. This is indicated by the somewhat irregular courses of Cedar Bayou and old Corpus Christi Pass. The long, narrow and low point of the barrier island north of such insecure passes is vulnerable to washovers. Several such washovers at Corpus Christi Pass were in existence in 1875 when the first U. S. Coast Survey chart was made and have been intermittently open since. Pass Cavallo, however, seems highly stable. This is because the simple boundaries of its large bay seem to keep the tidal delta from migrating down the coast. However, the channel is thrown against the southwest bank, Fig. 2, and may be eroding there.

NORMAL LOCATION & ORIENTATION OF INLETS ON TEXAS COAST

Our studies have been confined largely to the southwestern Texas coast where the meteorological conditions are relatively simple and are known to the writer. Here, it is established that:

1. The stable orientation of a tidal inlet channel on this coast is NNE-SSW, giving it a long diagonal course across the barrier island, making the tidal delta asymmetrical and causing the southern side of the island to be offset gulfward.

2. Tidal inlets migrate to the southwest until they have reached a position of stability, but may be driven on past it into a second stage of instability.

3. Tidal deltas and tidal inlets have a stable position southwest of an associated series of wide-mouthed bays.

4. Only one inlet will long be naturally maintained for a single bay.

5. Artificial cuts through the barrier island producing dog-legs in the inlet, and channels run across the direction of tidal currents and dominant wind-driven currents, have high maintenance costs that may be due in large part to failure of the channel (1) to lie in the direction of greatest flow and (2) to be located at the stable inlet position for the region.

From casual observations of maps of other coasts with barrier islands, it is believed that their inlets can likewise be analyzed by the application of the geological methods.

DETERMINING STABLE LOCATION & ORIENTATION ON OTHER COASTS

It has been shown that a scientific study of all the inlets and deltas of a more or less homogeneous coastal sector has determined the stable position and stable orientation for its tidal inlets. The same methods may be applied to other sections of the coasts of North America and to similar coasts elsewhere. Differences in detail will be found but it is

believed that most of the underlying principles have been determined, although not their full ranges of values.

WORLD-WIDE STUDY NEEDED TO COMPLETE KNOWLEDGE OF LAWS OF INLET DYNAMICS

Study of other coastal sectors with different fields of force and resulting different stable inlet positions should improve our knowledge of ship channel location. Such wide studies should permit general laws of inlets, tidal deltas and barrier islands to be set up, with a consequent improvement in the efficiency of engineering projects involving such coastal features.

POSSIBILITY OF REDUCING MAINTENANCE COSTS BY REORIENTING AND RELOCATING CHANNELS AND PASSES

REORIENTATION

To maintain a ship channel on an alignment contrary to the natural one would seem surely to result in excessive costs. If and where economically feasible, the pass through the barrier island should be re-oriented so that, if there is a strong offshore water movement from the bays, it may be utilized to the full to scour out the inlet channel. This may not be practicable, where as at Aransas Pass, too sharp a dog-leg would be produced in an existing long ship channel.

RELOCATION

The question of relocation of a ship channel would have to be carefully approached, because of the great expense to be incurred where there is a long channel with costly industrial installations. Studies of many ship channels through inlets, possible supplemented by model studies, might be made in seeking a set of cost figures.

LOCATION OF NEW SHIP CHANNEL THROUGH A BARRIER ISLAND

In this case, geology has an evident opportunity to assist the engineer.

If the hydrodynamics of a tidal delta has not been carefully studied by engineers or oceanographers, such a study should be made in the hope that the deep breaching of the front of the delta (outer "bar") or a modification of the position of the front so as to throw it into deep water might be made. The maximum depth of the foot of the barrier islands is shown by the offshore profile of the bottom. It is usually from 20 to 35 feet. This would indicate how far offshore control of the delta would have to be established for a particular depth of channel.

CONCLUSIONS

1. Tidal inlets used as oceanic outlets for ship channels may be in a naturally stable position or may tend to migrate.

2. The tidal inlet is part of a tidal delta. The delta is the product of a strong hydrodynamic field of force in which there is a meeting of (a) tidal currents, (b) wind-driven currents from inner waters, and (c) longshore sediment drifts and currents of the coastal lagoon and the oceanic shore.

3. Laws of the tidal delta and inlet for the central Texas coast - a unit geo-oceanographic environment - have been deduced from a geologic, meteorologic and oceanographic study.

4. Study of other homogeneous sectors of coasts with bays and barrier islands are needed to complete our knowledge of the basic laws for inlets and deltas. These studies should be geologic, geophysical and engineering in type.

5. Many ship channels use locations inherited from the days of shallow-draft vessels.

6. Locations and orientations of bay-port ship channels that go through tidal inlets should be inspected to see whether there is a heavy maintenance cost that may be due partly to courses run in opposition to dominant natural forces.

7. Engineering studies may be made of such inlets and their deltas to determine (a) the possible saving in maintenance costs if the channel were reoriented or relocated, and (b) further economics of such changes. These studies should include other coasts, as noted in Conclusion 4.

8. Among geological results of the present study is the suggestion that tidal deltas may be normally asymmetrical due to the barrier islands of which they are an integral part usually having a strong longshore sediment-drift.

9. There is normally a seaward offsetting of the shoreline of the barrier island at the down-stream bank of the tidal inlet relative to the dominant longshore drift.

REFERENCES

Bullard, F. M. (1942) Source of beach and river sands on Gulf coast of Texas, Bull. Geol. Soc. Amer., vol. 53, no. 7, pp. 1021-1043.

Price, W. A. (1947) Equilibrium of form and forces in tidal basins of coast of Texas and Louisiana, Bull. Amer. Assn. Petrol. Geol., vol. 31, no. 9, pp. 1619-1663. Corrections, no. 10, p. 1893.

PART 4
DESIGN OF COASTAL WORKS

Chapter 23

DESIGN AND PERFORMANCE OF SEA WALLS IN MISSISSIPPI SOUND

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The purpose of this paper is to describe briefly the various types of sea walls that have been constructed on Mississippi Sound, the performance of each type, and the conclusions with regard to sea wall design that can be drawn from this information.

DESCRIPTION

Mississippi Sound is a long narrow body of water that extends east and west a distance of about 75 miles along the coasts of Mississippi and Alabama. It is separated from the Gulf of Mexico by a series of barrier islands that lie about 10 miles offshore. See Figure 1. The water depths increase progressively from the mainland to the islands where they reach extreme values of 15 to 20 feet.

Tides in the sound are chiefly diurnal, having one high and one low in a 24-hour period. The mean tidal range is about 1.4 feet.

The mainland north of the sound is low and much of it is subject to inundation by storm tides. The natural shores are made of loose and highly erodible sand except in low marshy places where a black peat or muck is usually found. The offshore slope is very flat, the 6-foot depth contour generally lying more than a half-mile from the shore.

The coast is highly developed as a summer resort area and many hotels, summer homes, tourist cottages, and bath houses have been built. U. S. Highway 90, the principal coastal artery, runs along most of the shore.

HURRICANES

Unfortunately the sound lies in the path of tropical hurricanes and is subject to high storm tides and severe wave attack. These hurricanes have not only been destructive to beaches, bath houses, and sea walls, but have at times raised storm tides which inundated much of the land and carried the destructive action of the waves far inland.

The formation of high storm tides is favored by the shallowness of the water and by the progressively diminishing depths of water met by the wind-driven currents as they move inshore through both the Gulf of Mexico and the sound. When the generating wind blows from the southeast quadrant, the converging shoreline formed by the mainland and the delta of the Mississippi River acts to confine the wind-driven water and thus to further augment the height of the storm tide.
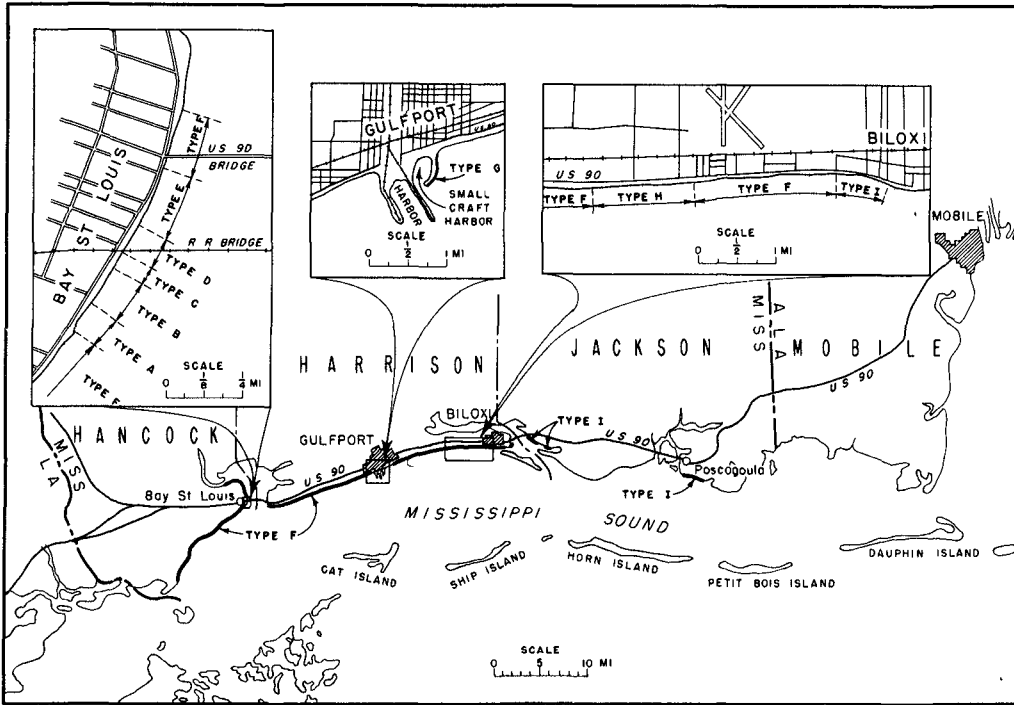
Fig. 1. The sea walls of Mississippi Sound.
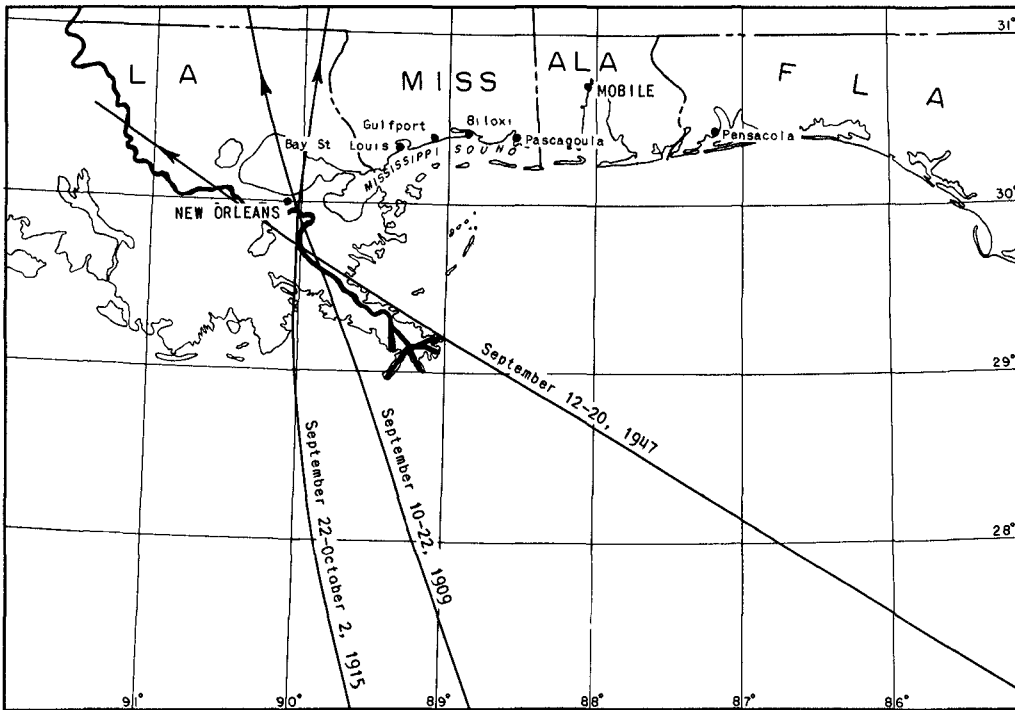
Fig. 2. Tracks for the hurricanes of 1909, 1915, and 1947.

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Figure 2 shows the paths followed by the three most destructive hurricanes that have occurred since 1900 on the coast under consideration. These are the hurricanes of 1909, 1915, and 1947.

It should be noted that the centers of these hurricanes all moved inland a short distance west of Mississippi Sound. Their extreme destructiveness on the north shore of the sound is due to this fact. Cline (1926) has shown that the strongest winds occur in the right rear quadrant of a hurricane and that the paths followed by the wind in that quadrant are not curved as one might expect in view of the counterclockwise circulation of a cyclone but have a fairly uniform direction which is roughly the same as the direction of advance of the center. Because of this uniformity in direction of the wind in the right rear quadrant the effective fetch is large. The length of the fetch and the unusual strength of the wind in that quadrant favor the generation of high storm tides and heavy wave action which affect primarily the coast to the right of the line of advance of the hurricane.

The hurricanes of 1909, 1915, and 1947 are said to have caused almost complete destruction of buildings in the unundated areas subject to wave action. It was largely the severe destruction caused by the hurricane of 1915 that awoke the people living along the coast to the need for protecting their shores. This was one of the most intense hurricanes in the history of the Gulf Coast. A storm tide of 9.6 feet was experienced at Gulfport and of 11.8 feet at Bay Saint Louis.

Following the 1915 hurricane the coast enjoyed a remarkable period of 32 years during which only minor hurricanes were experienced. This period was brought to a close by the extremely destructive hurricane of September 1947. Velocities in the right rear quadrant of this hurricane reached 100 mph and more. The direction of the wind as this quadrant lay over Mississippi Sound was roughly from east to west. The result was the creation of a storm tide that increased in height progressively from east to west. The following maximum storm tides occurred:

Location	Elevation, m.s.l.
Pascagoula, Miss.	7.68
Biloxi, Miss.	11.12
Gulfport, Miss.	14.03
Bay Saint Louis, Miss.	15.22

The sea walls were completely overtopped and numerous boats, including some of considerable size, were driven over the sea walls and some distance across the land. The waves passed over the sea walls to create havoc with buildings of all kinds landward of the walls. During the early part of the hurricane before the tide had reached the top of the sea walls and while it was still possible for an observer to remain near the walls it was observed in Harrison County that the angle of incidence of the approaching waves was roughly 45°. The same observers were unable to estimate the

height of the waves at that time. One observer between Bay Saint Louis and Gulfport who was able to see the waves from a second-story window of his home estimated that the height of the waves at the maximum tide was roughly 6 feet. From the storm tides, the wind velocities, and other known factors, A. H. Glenn (1951) has estimated the maximum breaker height at Bay Saint Louis to be about 8 feet.

THE SEA WALLS

The City of Bay Saint Louis, in Hancock County, between the years 1915 and 1920 constructed 5500 feet of sea wall along the shore adjacent to the city's main business district. The County of Hancock added 10 miles of sea wall during the period 1926 to 1928. Harrison County constructed 26 miles of sea walls during the period of 1925 to 1928. Jackson County completed about 3.5 miles of sea walls in 1929. In all there are now about 41 miles of sea walls along the north shore of Mississippi Sound.

The upright sea walls, types A, B, C, D, and E, shown in Figures 3 through 7, are of early construction. The overall length of these walls is 3050 feet.

The type A wall shown in Figure 3 was originally of brick masonry construction. The concrete face and the buttresses were added several years after the original construction. It is understood that the wall is equipped with tie backs but the position and type of construction of the tie backs are not known. During a minor hurricane in 1940 this wall tilted seaward and some backfill was lost. The wall was repaired and survived the 1947 hurricane with little or no damage.

The type B sea wall is shown in Figure 4. The depth to which this structure extends is not known. Neither is it known whether it is provided with tie backs. This wall survived the 1947 hurricane with little or no damage.

It is probable that the type D wall, shown in Figure 6, was originally a continuation of the type C wall which is shown in Figure 5. If this is correct, then the buttresses are later additions made to strengthen the wall. Along part of this wall some backfill has been lost and the wall has tilted landward. Where this has occurred severe cracking has taken place at the junction of the buttresses and the horizontal beam near the top of the wall. Along much of the type C wall the tie backs have failed and the wall has tilted seaward. Some backfill has been lost. It is apparent that corrosion of the tie backs has been a factor in causing this damage.

Much of the type E sea wall, shown in Figure 7, has failed by falling seaward. The concrete anchor piles were broken apparently because the backfill had been washed away from around them and the piles were not capable of functioning as cantilevers. The wall itself failed by breaking at the juncture of the wall and the footing.
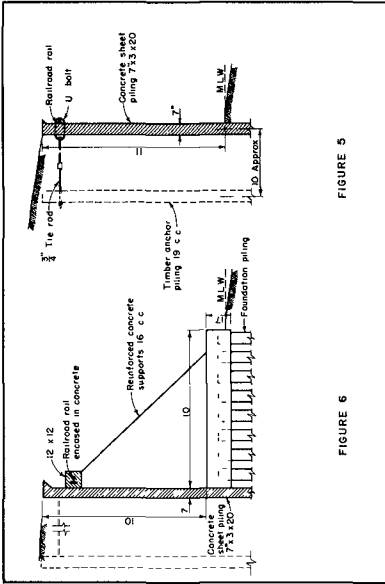
FIGURE 5

FIGURE 6

Fig. 5. Type C sea wall.
Fig. 6. Type D sea wall.

FIGURE 8

Fig. 8. Type F sea wall.
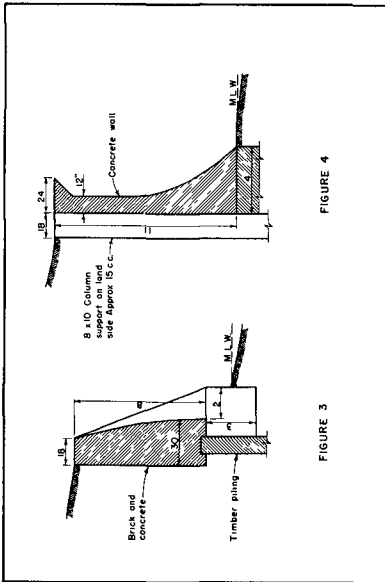
FIGURE 3

FIGURE 4

Fig. 3. Type A sea wall.
Fig. 4. Type B sea wall.
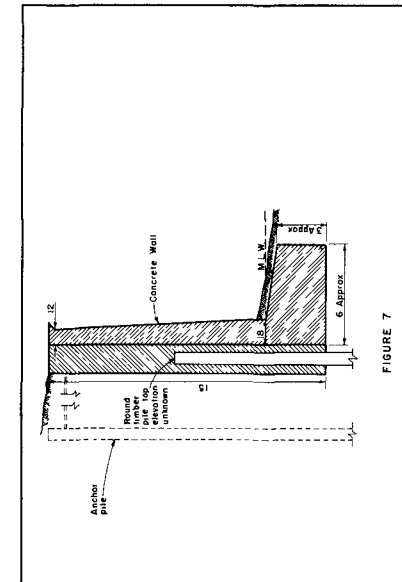
FIGURE 7

Fig. 7. Type E sea wall.

The type F or stepped sea wall shown in Figure 8 represents the predominant type of construction on the coast. There are about 10.7 miles of this type of wall in Hancock County and 24.0 miles in Harrison County. The number of steps varies from 3 to 10. When the number of steps is less than nine the middle row of pilings is omitted. The reinforcement shown in these sections is that used in the Harrison County walls which were designed by H. D. Shaw of Gulfport, Mississippi. The stepped walls in Hancock County are similar in design although timber piling has been substituted in some cases for the concrete piling.

Cracks appeared in many of the stepped slabs soon after they were completed. In the Harrison County walls a crack appeared at the intersection of the bottom step and the riser of the second step. A transverse crack from top to bottom also appeared at the center of many of the slabs. In some of the stepped slabs in Hancock County a crack appeared along the line of support of the middle row of piles.

To explain these cracks it should be considered that the joints between the stepped slab and its supporting piles are rigid or nearly so. The maximum bending moments produced by differential settlement or other causes therefore occur in the vicinity of these joints. Some cracks were of sufficient size to permit sea water to reach the reinforcement. The result at many points was rust bleeding, expansion of the reinforcing steel by corrosion, and further cracking and breaking of the concrete.

The stepped sea walls also suffer from the defect of not being sand tight. The storm sewers that pass through the sea walls were placed directly on sand. Through differential settlement or otherwise the joints in the sewer pipes failed and permitted the escape of backfill. The concrete sheet-piling cut-off wall also permitted the escape of sand. The sheet piles used in the cut-off wall were provided with tongue and groove joints only for the lower parts. See Figure 8. Above that, two grooves were provided, the theory being that the cavity formed by two adjacent grooves would be filled with grout and thus rendered sand tight. During construction it was found that the cavities filled with sand and could not effectively be grouted; consequently the cut-off wall could not be made sand tight. The loss of backfill caused the further deterioration of the storm sewers and the collapse of the sidewalks adjacent to the sea wall.

During the 1947 hurricane the stepped sea walls failed in a number of places. Available evidence indicates that in most cases the slabs moved seaward when they failed. What probably happened is that the alternating downward and upward forces exerted by the peaks and troughs of the waves as they reached the wall produced continually reversing stresses in the wall which caused the concrete in the vicinity of the supporting piles to crack and crumble until finally the joints failed entirely. The slab was then lifted from its supports and moved seaward. As a general rule the sheet piling cut-off wall tilted seaward but the bearing piles broke away from the slab without tilting.
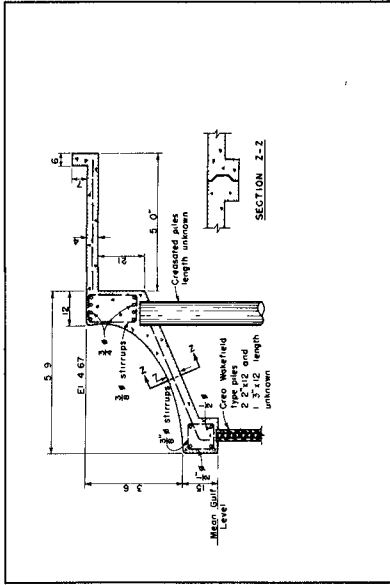
Fig. 10.

Fig. 9. Type G sea wall.
Fig. 10. Type H sea wall.
Fig. 11. Type I sea wall.
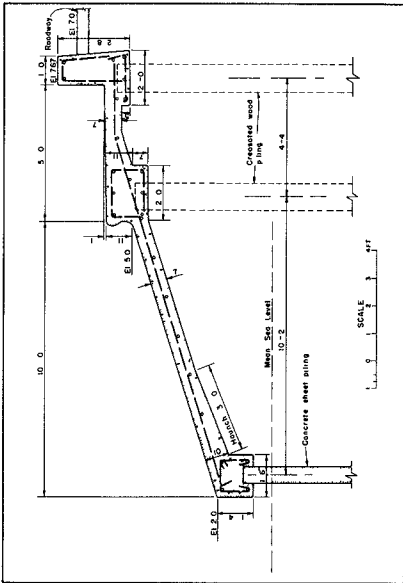
Fig. 9.
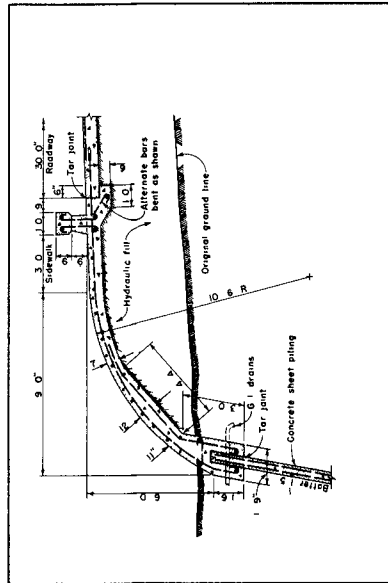
Fig. 11.

In connection with the stepped sea walls it should be stated that the number of sections of the wall that failed constitutes but a small percentage of the total number along the coast. In Hancock County about 1.8 percent of the stepped walls was lost and in Harrison County about 0.4 percent.

There are about 2,500 feet of the type-G sea wall shown in Figure 9 on the east and south sides of the causeway at the Gulfport small-craft harbor. Several sections of this sea wall were lost during the hurricane. These sections were lifted from their supporting piles by hydrostatic uplift and moved seaward. The concrete slab broke at the point where it meets the cap on top of the sheet-piling wall.

There are about 1.3 miles of the type H concave sea wall shown in Figure 10 at Biloxi in Harrison County. Several sections of this wall were lifted from its supporting piles by hydrostatic pressure and moved seaward during the hurricane.

The type I sea wall, shown in Figure 11, has a convex face instead of the usual concave face. There is about 0.65 mile of this wall at Biloxi in Harrison County, 1.6 miles at Ocean Springs, and 2.0 miles at Pascagoula, both of which are in Jackson County. This wall depends on backfill for all of its support except that provided at the toe by the sheet-piling cut-off wall. It suffers from the defect that it facilitates rather than hinders the movement of waves over the wall and onto the adjacent roadway. This wall has lost little or no backfill and there is no evidence of settlement in either the wall or the adjacent roadway. It weathered the 1947 hurricane with practically no damage. It should be mentioned, however, that all three of the locations where this wall is found are in relatively sheltered positions where the wave action during the 1947 hurricane was not as severe as it was elsewhere in the sound.

DISCUSSION AND CONCLUSIONS

It should be realized that much of the natural ground that lies landward of the sea walls on Mississippi Sound is below the level of the more extreme storm tides that have occurred in the sound. For that reason damage due to inundation is inevitable or at least cannot be prevented by means of sea walls. However, sea walls of suitable height can serve the purpose of intercepting the waves and thus protecting the property behind the walls from the destructive action of the waves.

The determination of the height of a sea wall on a coast subject to high storm tides is largely a matter of economics. It may not be possible to justify a sea wall of sufficient height to prevent overtopping of the wall at all times. The probable frequency of occurrence of storm tides and waves severe enough to pass over the wall and cause damage to the property landward of the wall must be considered. As formulas presently available for estimating wind tides and wave heights in shallow bodies of water are somewhat unreliable, use should be made of past records insofar

as possible. In this connection it may be stated that comprehensive studies of wind tides and waves on Lake Okechobee in Florida are now being made by the Corps of Engineers and more reliable formulas should become available in the future.

In evaluating the effectiveness of a sea wall against an assumed combination of storm tide and wave height it may be assumed roughly that, if the top of the wall is at a level equal to or greater than the storm tide plus the wave height, little wave damage will occur to property landward of the sea wall. It is assumed of course that adequate protection has been provided for the backfill which may otherwise be seriously damaged by spray thrown over the wall.

Much of the damage that has occurred to sea walls on Mississippi Sound and elsewhere has been due to the loss of backfill. Backfill may be lost through defective joints or drains. It may also be lost if the wall is overtopped or if it is flanked at either end.

To prevent the loss of backfill extreme care should be taken to obtain tight joints between the sheet piles and elsewhere in the sea wall. If there is any doubt as to the sand tightness of the joints, graded filters should be provided behind them. Graded filters should also be provided behind all drains and weep holes. Where reinforced-concrete sheet piling is used for a cut-off wall the tongue-and-groove joint should extend the full length of the pile.

If it is impracticable or uneconomical to construct a sea wall completely free from overtopping, the fill should be protected with a pavement. If only overtopping by spray is anticipated a water-tight pavement of concrete or asphalt with a well-anchored watertight joint at the wall will be satisfactory. Recurving the face of the sea wall seaward at the top to direct the spray toward the sea will also be helpful. If, however, the wall is designed to permit overtopping by storm tide and waves then a water-tight pavement will not be satisfactory because it will be subject to the destructive action of hydrostatic uplift as the trough of a wave passes over it. It is unlikely that weep holes with graded filters can be provided with sufficient capacity to relieve this uplift. A more reliable solution would be to provide a riprap pavement placed on a gravel blanket.

If the end of a sea wall does not abut another sea wall or similar structure, a wing wall should be provided to prevent flanking. A vertical bulkhead will ordinarily serve for this purpose. The wing wall should extend inshore a distance greater than the maximum recession likely to occur on the adjacent unprotected shore. The wing wall should be backfilled on both sides and the fill, together with part of the surrounding ground, should be covered with a riprap pavement lying on a gravel blanket.

If the sea wall is long there is some advantage in dividing it into separate compartments by means of transverse diaphragms. In this way the

loss of backfill will be localized if a break occurs in the sea wall. In view of the frequency with which backfill from behind sea walls has been lost in the past and in view of the difficulty in designing a sea wall entirely secure against such loss, it is desirable to design sea walls to be as nearly safe structurally as possible without the backfill. This is particularly important in the case of a sloping or stepped sea wall supported by bearing piles, as a slight settlement of the fill is inevitable and no pressure between the slab and the fill can be relied upon to support the wall. Where tie-back anchors are used, the anchors should be piles driven to ample penetration and the tie-backs should be capable of functioning in compression as well as in tension.

It is desirable to maintain a beach of adequate height in front of a sea wall. In addition to having both a recreational and an aesthetic value such a beach adds to the stability of the sea wall by increasing the soil pressure against the toe and the cut-off wall. It retards or prevents the escape of backfill if the wall is not sand tight and provides protection against wave attack by forcing the waves to break before they reach the wall. However, in the case of a storm tide that raises the water level above the beach, this protection is partly lost.

With a view to obtaining such a beach, sand is now being pumped in front of the sea walls in Harrison County by means of hydraulic dredges. This is being done as part of a cooperative project between the County of Harrison and the Corps of Engineers. It is hoped that information obtained from this operation will be helpful in planning similar projects elsewhere.

The action of waves on the sea walls of Mississippi Sound has not been observed during the fury of a severe hurricane. Nor is it likely that the mere visual observation of such wave action would throw a great deal of light on the complex phenomena involved in the reflection or breaking of waves against the many varieties of sea walls on this coast. These phenomena together with the pressures that they create against the sea walls are therefore beyond the scope of the present paper. They are appropriate subjects for investigation in model laboratories.

The importance of hydrostatic forces behind a sea wall should be emphasized. Available evidence indicates that these forces have been the dominant factor in causing sea-wall failures on Mississippi Sound. The need for providing tie-backs and anchor piles behind vertical walls capable of functioning after the backfill has been lost has already been mentioned. In the case of stepped, sloping, or curved walls resting on bearing piles the slabs should be securely fastened to the supporting piles to resist the uplift due to the hydrostatic forces under the slabs. Since these slabs are subject to alternating downward and upward forces as the peaks and troughs of the waves reach the wall, they should be reinforced to withstand the resulting bending moments. To resist the alternating horizontal thrust against the wall the bearing piles should be of sufficient strength and penetration to function as cantilevers after the fill has been lost.

To prevent the corrosive action of the sea water on the reinforcing steel, the concrete should have the lowest porosity attainable and should cover the steel by not less than three inches. At this point the author would like to suggest that there may here be a fertile field for the use of prestressing as a means of keeping the outer face of the sea wall under compression and thus preventing the passage of sea water through haircracks to the reinforcing steel.

ACKNOWLEDGMENT

REFERENCES

Cline, I. M. (1926). Tropical Cyclones, The MacMillan Company, New York.
Glenn, A. H. (1951). Personal communication to author.

Chapter 24

HISTORY OF THE
GALVESTON SEA WALL

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Galveston District, Corps of Engineers
Galveston, Texas

Galveston Island, one of the long narrow barrier beaches that fringe the Gulf Coast of Texas, is a low, sandy formation about 28 miles long and from $\frac{1}{2}$ to 3 miles wide. In its natural state the Gulf shore was bordered by an area of sand dunes rising to heights of 12 to 15 feet above the natural surface of the island. The availability of deep water along the bay side of the island led to the early development of the city of Galveston on the east end of the island. In its early days the city was protected from hurricane tides by the sand dunes along the Gulf front. The rapid development of the city in the latter part of the 19th century, especially its increasing importance as a summer resort, lead to the removal of the sand dunes along the beach front for fill and to permit easy access to the beach. Without the dunes the city was unprotected from the fury of the hurricanes. The danger to the city was realized by a number of persons, and several plans for storm protection had been developed; however, because of financing difficulties and general public apathy none of these plans was carried out. Figure one shows a map of Galveston Island with development as it was in 1900.

The resort city of about 38,000 persons was exposed to the havoc of the hurricane of September 8, 1900. The hurricane winds of this storm and accompanying 15-foot tide that swept the city caused property damages reported to be over $25,000,000, and a loss of more than 6,000 lives. The damage along the Gulf front was complete: In about 1,500 acres along the Gulf front over 2,600 houses were destroyed and up to 300 feet of land were lost by erosion of the shore.

The citizens of Galveston responded to the destruction of the city with remarkable energy and fortitude and within four years had erected a barrier to the sea that was to save the city from further devastation within 15 years and which still makes Galveston a comparatively safe place in which to live.

About one year after the storm the City Commission of Galveston and the County Commissioners Court of Galveston County appointed a board of engineers to report on means for protecting the city. The board was composed of Brigadier General H. M. Robert, United States Army, retired; and Messrs. Alfred Noble and H. C. Ripley. The board was directed to report on the following:

1st. The safest and most efficient way for protecting the city against overflows from the sea.
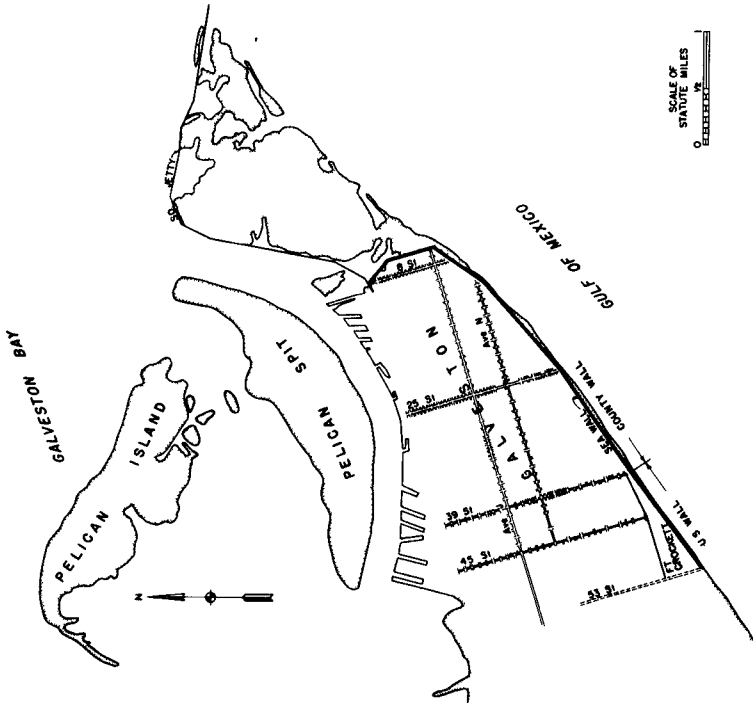
Fig. 2.
Galveston Island showing initial sea wall construction by Galveston County and the United States.
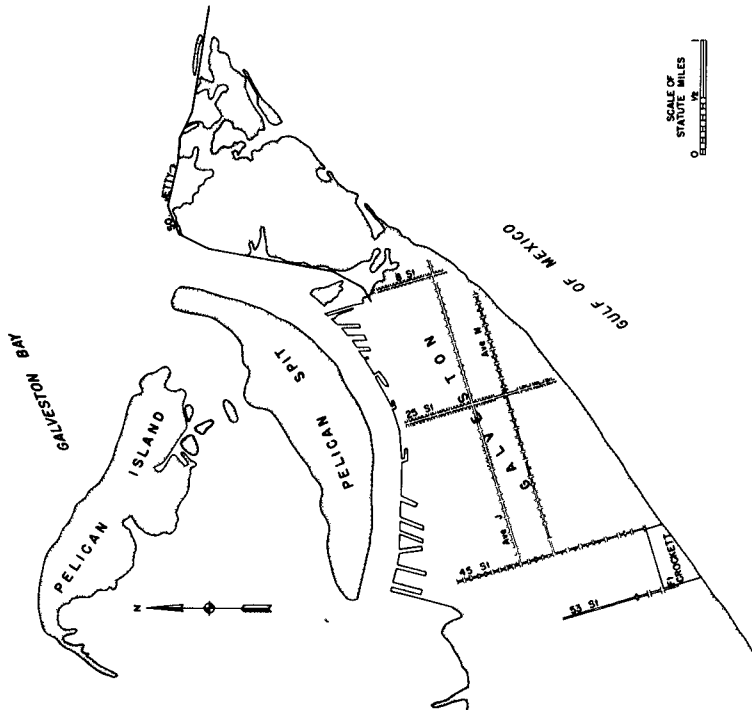
Fig. 1.
Galveston Island about 1900.

2nd. Elevating, filling, and grading the avenues, streets, sidewalks, alleys, and lots of the city so as to protect the city from overflow from the waters of the Gulf, and to secure sufficient elevation for drainage and sewerage.

3rd. A breakwater or sea wall of sufficient strength and height to prevent the overflow of and damage to the city from the Gulf.

The Roberts Board submitted its report on January 25, 1902, recommending in general the following plan for protection of the city:

a. A solid concrete wall, over 3 miles long, connecting with the south jetty near 8th Street; thence to 6th Street and Avenue D; and thence on 6th Street across the island to the beach and down the beach as far as 39th Street. The top of this wall to be 17 feet above mean low water. The location of this sea wall would be landward of the highwater line generally at about the 3-foot contour as shown on figure 2.

b. The raising of the city grade to 8 feet at Avenue A, 10 feet at Broadway, 12 feet at Avenue P, and continuing this slope to the sea wall, which corresponds to a rise of one foot in 1,500 feet from the bay toward the Gulf.

c. The making of an embankment on top of this fill adjacent to the wall, and rising to a height of 18 feet above low water at a distance of 200 feet from the wall, thence sloping down on a grade of 1 in 50 to the surface of the fill. The top of this embankment for 35 feet from the sea wall to be protected by a brick pavement and 60 feet farther by Bermuda grass. A section of the structure proposed by the Roberts Board is shown on figure 3a.

The design of the sea wall provided for a concrete gravity section 16 feet wide on the base at elevation 1 foot above mean low water and 5 feet wide on top at elevation 17 feet above mean low water. The sea face of the concrete sea wall to be curved so that its upper portion will be vertical, to give the wave an upward direction and prevent, to a great extent, its running up and over the embankment behind the wall. The wall would be founded on piles and protected from undermining by sheet piling and a layer of riprap 27 feet wide and 3 feet thick extending outward from the toe of the sea face of the wall. The riprap to be deposited so that when the work is finished the larger stone will be on the surface with interstices closely filled with smaller pieces, the object being to present a surface as smooth and resistant to wave action as practicable without incurring great expense in placing the stone.

The cost of the sea wall was estimated at a total of $1,295,000 for the 17,593 feet of sea wall. This portion of the sea wall was constructed by Galveston County, beginning in October 1902 and completed in July 1904 at an actual cost of $1,581,673.30. The structure was built generally in accordance with the plans of General Robert, except that the embankment
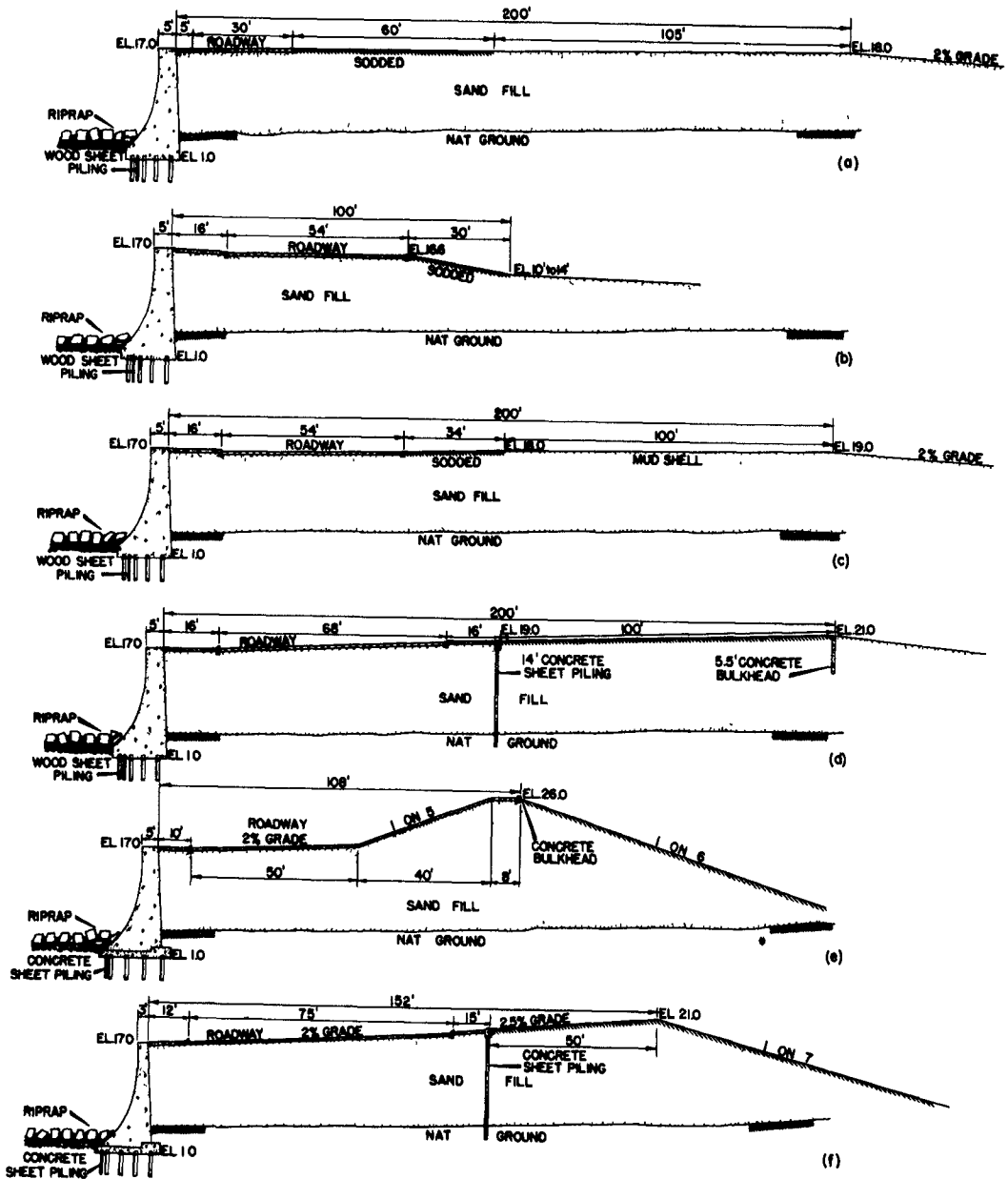
Fig. 3. Cross sections of the sea wall.
   a. As proposed by the Roberts Board.
   b. As initially built by Galveston County.
   c. As modified after the 1909 storm.
   d. As modified after the 1915 storm.
   e. As built in front of Fort San Jacinto.
   f. Authorized 3-mile west extension.

behind the concrete section was built only 100 feet wide and to a maximum elevation of 16.6 feet. The wall as built is shown on figure 3b.

While the county sea wall was being constructed the U. S. Congress authorized the construction of a sea wall of similar design along the front of the Fort Crockett Military Reservation. This reservation is located between 39th Street and 53rd Street, beyond the end of the county sea wall. The Fort Crockett sea wall is 4,935 feet long and ties in three concrete gun emplacements on the reservation. It was constructed immediately following completion of the county wall at 39th Street, being started in December 1904 and completed in October 1905. The cost to the United States was $295,000, exclusive of the fill behind the wall.

The concrete section of the Fort Crockett wall is similar to the county wall as recommended by the Roberts Board. Behind the wall the entire reservation was filled to a grade of 17 to 18 feet, and 34 feet of the fill adjacent to the concrete wall was protected with paving.

The sea wall was soon subjected to a minor test, when in 1909 an intense hurricane of small diameter crossed the Texas coast about 45 miles southwest of Galveston. The accompanying storm tide reached about 6.6 feet above mean low water. Considerable quantities of water were thrown over the sea wall and drained across the fill into the city. The slope of the sand embankment behind the wall was considerably scoured by the flow of water and some of the roadway paving was undermined. The riprap along the toe of the wall was lowered somewhat and in a few places the wooden sheet pile cut-off wall was exposed.

The toe of the wall was repaired by placing sand and riprap to bring the riprap up to grade. As a result of the damage to the embankment caused by this storm it was decided to repair the embankment behind the wall and extend the fill to a crest elevation of 19 feet at a distance of 200 feet from the face of the concrete section as originally proposed. The section as repaired and modified after the 1909 storm is shown on figure 3c.

In 1913 a report was prepared at the request of Congress by a special board of engineers for rivers and harbors, which considered the question of extending the sea wall eastward from 6th Street to Fort San Jacinto in the east end flats. This report pointed out the danger of storm erosion cutting a channel across the island in this area, which would breach the jetty and result in extensive shoaling in the Galveston channel along the bay side of the island. The board considered this danger sufficient to warrant preventive measures and recommended that the sea wall be extended across the east end flats to protect the harbor from blockade by storms and to permit expansion of the harbor facilities. The recommended extension from 6th Street to the first battery at Fort San Jacinto had a total length of 10,300 feet of which
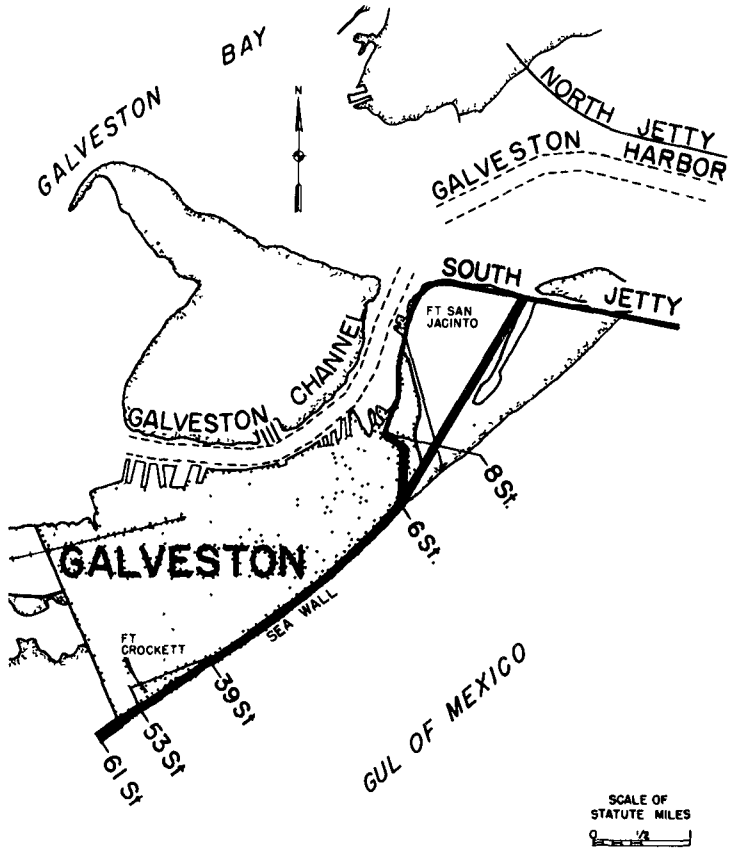
Fig. 4. Galveston Island showing sea wall
extension to the south jetty and to 61st Street.
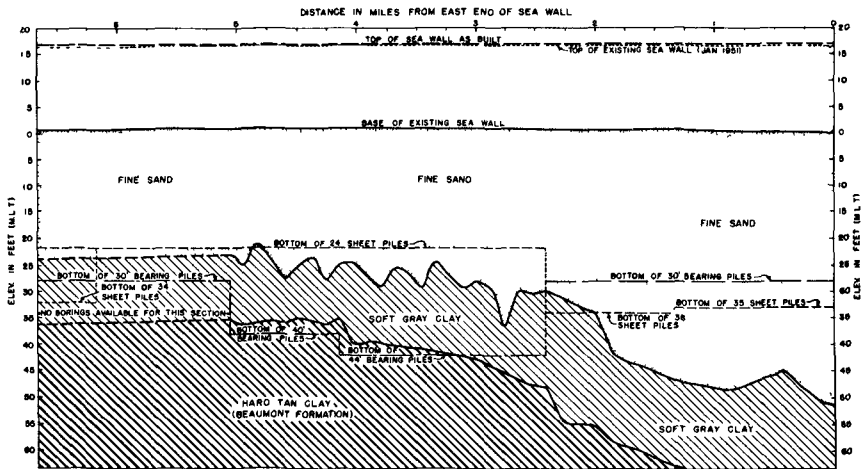
Fig. 5. Soils profile along the existing sea wall.

3,300 feet would be built by the local interests, city or county, and 7,000 feet on the Fort San Jacinto Reservation would be built by the United States. This proposed extension is shown on figure 4. The design of the sea wall extension was the same as the completed sea wall with the embankment behind the wall 200 feet wide from the edge of the wall to the crest of the fill.

In 1915 the sea wall was subjected to its most severe test, in fact probably as severe a test as it will ever experience, for the tropical cyclone that crossed the Texas Coast on August 16, 1915, about 30 miles southwest of Galveston, was a major hurricane fully as severe as the storm of 1900 that wrecked such havoc in the city.

The sea wall proved the adequacy of its design in protecting the city from a repetition of the damage it had experienced in 1900. In Galveston the loss of life in 1915 was only 12 and the total property damage was estimated at $4,500,000, both many times less than that caused by the 1900 storm.

The 1915 storm was accompanied by a tide that reached nearly 14 feet above mean low water and wave crests that are estimated to have reached a maximum height of about 21 feet. The storm was of particularly long duration, being several times that of the 1900 storm, and the storm tides inundated the city for over 40 hours.

The heavy waves caused considerable scour along the foot of the sea wall. The riprap apron was undermined in many places, dropping as much as 3 feet below the toe of the wall, and exposing the timber sheet pile cut-off wall. This was of particular concern since the untreated timber was exposed to teredo damage. The exposed reaches were filled with sand and riprap as rapidly as possible. Over 24,000 tons of riprap were placed along the toe of the wall to repair the storm damage.

The most extensive damage to the sea wall was the erosion of the embankment back of the wall by the great quantity of water that was thrown across the wall. One observer reported that water appeared to be coming over the wall in a continuous sheet about two feet deep. The embankment was scoured out and the pavement destroyed completely from 6th Street to 18th Street. Several houses and buildings near the sea wall were undermined by the scour and destroyed. Between 18th Street and 21st Street the embankment was protected from scour by buildings. West of 21st Street the embankment was washed down from 7 to 8 feet and the brick paving was damaged, and from 39th Street to 43rd Street, in front of Fort Crockett, the embankment with road and sidewalk paving was washed back into the city. There was considerable scour from 43rd Street to the end of the wall at 53rd Street.

The concrete section of the sea wall was damaged to the extent of two small chips of about two cubic feet each near 39th Street. This was caused

by a four-masted schooner which was blown over the wall during the height of the storm while dragging two anchors. The anchors caught on the toe of the wall and the schooner pounded to pieces on top of the wall. Fragments of the hull, masts, and cargo were scattered over the west end of the town.

Before the storm there was a beach generally along the Gulf side of the wall, as much as 300 feet wide in places. After the storm the beach had completely disappeared and there was a depth of 3 to 4 feet at the toe of the riprap. The beach sand was in a bar deposited several hundred feet off-shore from which a large quantity gradually moved inshore but the beach has never built up above low tide to any extent since the storm.

Because of the extensive damage to the sea wall, the county requested General Robert to review the problem and report on a plan to furnish further protection against hurricanes. His recommendations were that the paving on top of the embankment be extended to a width of 100 feet; that a reinforced concrete sheet pile bulkhead be placed along the land side of the pavement, with a top elevation of 19 feet; and that the embankment rise in another 100 feet to an elevation of 21 feet at 200 feet from the wall.

This work was done immediately and in addition a small concrete bulkhead, 1 foot thick and 5 feet high, was constructed at the crest of the embankment. The sand fill beyond the paving was protected by sodding or by a cover of shell.

Figure 3d shows this modification.

On July 27, 1916, Congress authorized the east extension of the sea wall from 6th Street to Fort San Jacinto as recommended by the board of engineers in 1913. This extension is shown on figure 4. Work on this extension was begun on June 20, 1918. The work was very much delayed by wartime labor shortages and lack of materials because of embargoes on railroad cars. The extension was about half completed, having reached the edge of an old borrow pit about 200 feet wide and $2\frac{1}{2}$ feet deep from which about 3,000,000 cubic yards of material had been removed for the Galveston grade raising, when the severe hurricane of September 13-14, 1919, occurred. The borrow pit had been closed with a wood sheet pile bulkhead and work of constructing the sea wall across the area was in progress when the storm occurred. This storm passed about 180 miles south of Galveston but it caused a tide of 9.0 feet above mean low tide and winds of about 60 miles an hour at Galveston.

The high tide covered the east end flats with several feet of water and caused strong currents around the end of the completed wall that scoured the borrow pit to a width of about 2,000 feet with a maximum depth up to 19 feet and an average depth of about 8 feet. It was necessary to place about 250,000 cubic yards of sand fill in the scoured

channel. The fill was slow in draining and considerable difficulty was experienced in constructing the wall across this reach.

The volume of water thrown over the sea wall during this storm was not large and there was little damage behind the wall except in the section at the Fort Crockett Reservation that had been washed out in the 1915 storm. Here the embankment had been replaced but had not been paved and sodded. The sand fill was scoured to depths of 2 feet to 12 inches between the wall and the sheet pile bulkhead 100 feet from the wall.

West of 39th Street the riprap at the toe of the wall was deficient in quality and quantity, being small in size and partly of sandstone. This rock was scattered and lowered for a distance of several hundred feet and in places the wood sheet piling were exposed. About 6,000 cubic yards of rock were required to repair this section of the wall.

The east extension of the wall to the battery at Fort San Jacinto was completed in March 1921. In the last 4,660 feet of this wall reinforced concrete sheet-piles were used for the cut-off wall under the toe of the sea wall, and the riprap apron was omitted because of the protection afforded by the wide foreshore in front of the wall.

A somewhat different design was used for the embankment behind the sea wall across the San Jacinto Reservation. Here the embankment behind the wall has a 10-foot walk and 50-foot roadway that slope up on a 2 percent slope and then the embankment rises on a 20 percent slope for 40 feet to a crest of the fill at elevation 26 feet. The crest is 8 feet wide and there is a concrete cut-off wall along its landside that extends one foot above the top of the fill. Behind the cut-off wall the fill has a 1 on 6 slope to the natural ground at a distance of about 250 feet from the face of the concrete wall. The front slope and crest of the embankment are paved. This embankment is designed to prevent any overtopping of the fill by storm waves. A section of this wall is shown on figure 3e.

A further extension of the sea wall eastward across the Fort San Jacinto Reservation to the south jetty was authorized by Congress in 1922, and was constructed between May 1923 and January 1926. This extension of 2,860 feet in length was of the same design as the completed wall on the reservation.

In 1926, Galveston County constructed a west extension of the sea wall from 53rd Street to 61st Street. This section, 2,800 feet in length and of the same design as the county wall in front of the city, was completed in June 1927.

The completion of this section brought the sea wall to its present condition. The total length of sea wall constructed is 38,490 feet, or

7.29 miles, of which 23,755 feet was constructed by Galveston County and 14,735 feet was constructed by the United States. The effective length of sea wall along the Gulf front is 6.64 miles. The cost of the sea wall totaled $6,130,000, and the cost per foot varied from $90.00 a foot for the first construction in 1902 to $200.00 per foot for the construction in 1926.

A profile of the sea wall from 61st Street to the south jetty is shown on figure 5. This profile shows the subsurface strata which consists of about 25 feet of sand underlain by 10 feet of soft clay below which is the heavy clay of the Beaumont Clay formation. The east end of the profile shows the erosion of the gorge of Galveston pass. The bearing piles under the wall are generally seated in the clay strata, except in the east end where the depth to clay is too great, and the section west of 39th Street where short piles were used.

The profile of the top of the wall, in figure 6, shows the extent of settling that has occurred in the wall. The oldest part of the sea wall has settled least because of the use of longer bearing piles, the tips of which extend into or near the Beaumont Clay formation. Settlement of the wall has been caused, mainly, by consolidation of the soft gray clay beneath the tips of the piles that do not reach the Beaumont Clay. Settlement has been very uniform at any one locality along the sea wall and no cracking or shifting of the monoliths is evident. Settlement has been continuous since the sea wall was constructed and apparently is continuing. The greatest settlement occurred in the east end of the sea wall which was constructed over recent deposits. Here the soft gray clay layer is much thicker and it appears that this area was the entrance to Galveston Bay until the last few centuries. Settlement ranges from 1.45 feet near the east end of the wall to a mere 0.1 foot near the central portion of the County wall about at 27th Street which was built in 1902-04.

Concurrently with construction of the county sea wall the city of Galveston undertook extensive grade raising in the city behind the wall. This work started in 1903 and continued at intervals through 1948, at which time all of the area within the city except a 10,200-foot section north of Broadway and a narrow strip along the west city limits had been filled. The present grade of the city slopes uniformly from an elevation of 14 feet on Avenue T behind Fort Crockett to 10 feet at Broadway and 8 feet along the channel. The cost to the city for grade raising amounted to about $6,000,000, of which about $4,000,000 was received from State taxes remitted to the city for this purpose. The Fort Crockett Military Reservation was filled at the time the sea wall was constructed. The San Jacinto Military Reservation has been filled to a considerable extent with spoil from channel dredging.

The sea wall has not been subjected to a severe test since 1919. The highest storm tide since then was 7.7 feet above mean low tide which occurred in 1942. Four other storms caused tides of between 6.0

and 7.4 feet at Galveston. The only damage to the sea wall from these
storms was loss of beach sand and slight lowering of the riprap at the
toe of the wall. The roadway ramps at the south jetty, 6th Street, and
61st Street were usually damaged and there was generally some erosion
around the west end of the wall. Repairs consisted of rebuilding the
ramps and replacing riprap as required. Data on the cost of maintenance
of the sea wall are not available. Normal maintenance other than repair
of storm damage has consisted principally of repairing and repaving the
roadway on the embankment behind the sea wall.

The sea wall between 10th Street and 53rd Street as originally con-
structed was located so that there was an appreciable beach, up to 300
feet wide in places, on the seaside of the wall. The storm of 1915
washed away practically all of this beach. Some of the beach materials
were returned to the beach within a short period; however, several suc-
ceeding cycles of erosion and accretion resulted, in 1934, in lowering
the beach and in recession of the shore line until it generally coin-
cided with the toe of the riprap along the sea wall. There was danger
that further loss of sand from in front of the sea wall would expose
the untreated wooden piling under the sea wall to destruction by teredo.

A cooperative beach erosion control survey, made in 1934 by the
Beach Erosion Board, Corps of Engineers, concluded that the sea wall
could best be protected and a beach for recreation be provided by the
construction of a system of groins from 12th Street to 61st Street.
The Board further concluded that a groin system might not be filled by
natural action and that artificial replenishment of the beach materials
might become necessary. Construction of the proposed groin system was
authorized by Congress in 1935, and a system of 13 groins, each 500
feet long and 1,500 feet apart, between 12th Street and 59th Street,
was constructed from 1936 to 1939. The groins have accumulated con-
siderable quantities of beach materials, most of which is below mean
low tide, and have kept the toe of the sea wall well protected. Excess
loss of beach materials during hurricanes has prevented accumulation
of sufficient materials to provide a suitable beach for recreation.

The protection that the sea wall afforded the city of Galveston
encouraged development of the city. The protected area became densely
occupied with houses and at present there is little undeveloped land,
protected by the sea wall, available for expansion of the city. Develop-
ment has taken place toward the southwest beyond the sea wall, despite
the danger of destruction by hurricanes. In order to afford protection
to this area, Congress in 1950 authorized construction of a 3-mile south-
west extension of the Galveston sea wall, similar in design to the exist-
ing wall. The riprap at the toe of the wall is 40 feet wide and the em-
bankment behind the wall rises to an elevation of 21 feet at a distance
of 155 feet from the face of the wall. The top of the wall is reduced
to a width of 3 feet. A section of the proposed wall is shown on figure
3f. The authorization provides that local interests contribute
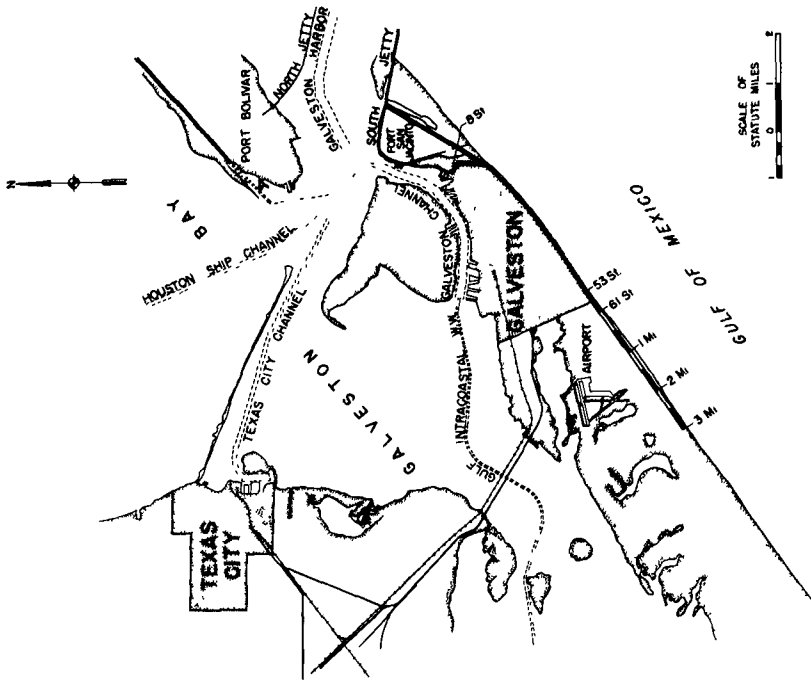
Fig. 7. Galveston Island showing authorized 3-mile west extension of the sea wall.
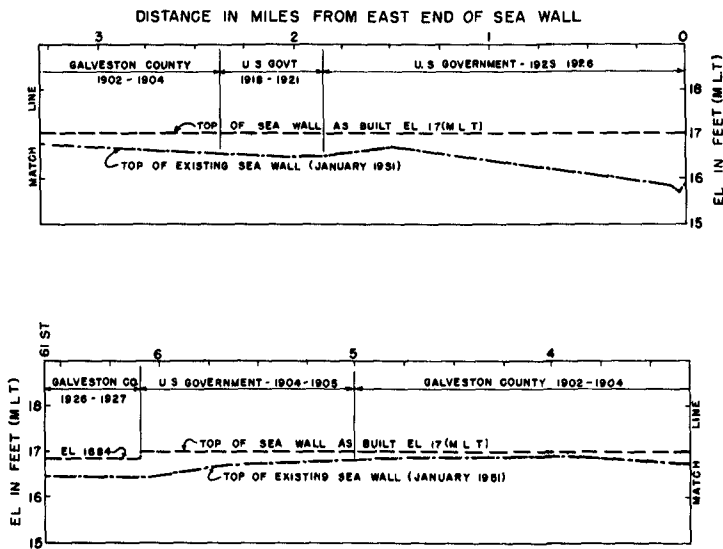
Fig. 6. Profile of the top of the existing sea wall showing settlement.

$2,870,000 toward the cost of the project, estimated at about $9,000,000.
The estimated cost is about $550.00 a linear foot of wall. No Federal
funds have been appropriated by Congress; however, Galveston County has
contributed its share to the Corps of Engineers and construction of 5,151
feet of the extension is now under way. (Fig. 7).

This culminates the history of the Galveston sea wall.

Chapter 25

MOBILE BREAKWATERS

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INTRODUCTION

A mobile breakwater may be defined as a structure or device which combines the ability to appreciably reduce the height of ocean waves in its lee with a degree of mobility sufficient to permit its ready transportation for considerable distances and its speedy installation when arrived at the site. Such a device would find application wherever wave protection is necessary for but limited periods, as in offshore drilling operations, or where an installation is required to be completed in a very short time, as in amphibious military operations.

Several attempts have been made in the past to achieve the requirements of a mobile breakwater, some with a fair degree of success. A considerable effort in this direction was expended during World War II, which resulted in the "Phoenix" and "Bombardon" breakwaters used in the Normandy invasion. For the past several years, the Hydraulic Structures Division of the Hydrodynamics Laboratory, California Institute of Technology, has been engaged in a general study of the mobile breakwater problem under the sponsorship of the Bureau of Yards and Docks of the Department of the Navy. This study has resulted in a better understanding of the principles which must govern any mobile breakwater, and has provided the systematic analysis of various projected mobile breakwater schemes.

THEORY OF BREAKWATER ACTION

The action of any breakwater may be considered in terms of basic wave processes. The theory of breakwater action therefore includes all wave processes which may result in wave height attenuation. The processes of wave refraction, wave interference, wave dissipation, and wave reflection are considered herein. These processes may be briefly described as follows:

Wave refraction is a process resulting from changes in wave velocity, and can result in either an increase or decrease of wave steepness. In the first case, wave breaking with resulting energy loss may occur, and in the second case the desired decrease in wave disturbance is obtained directly.

Wave interference is a process involving the vector addition of the particle velocities of two or more superimposed wave trains. The wave height may be increased or decreased, depending on the direction and phase of the interfering wave trains.

Wave dissipation is the conversion of wave energy into heat energy through the action of frictional forces.

Wave reflection, in the ideal case, is a change in wave direction without energy loss, hence may be likened to perfectly elastic impact. The change in wave direction is determined by the equality of angle of incidence and angle of reflection.

The application of these four processes to the problem of mobile breakwater design will now be considered in more detail.

WAVE REFRACTION

Refractive processes are those associated with changes in wave velocity. The familiar case of water wave refraction is that due to changes in bottom topography (Johnson and O'Brien, 1946), the non-uniform depth corresponding to non-uniform wave velocity in the area considered. The basic method of analysis of such problems consists of plotting the position of successive wave crests by consideration of the local wave velocity at each point, followed by construction of orthogonals to the wave crests. The energy transmitted per unit time, or wave power

$$P = E C_g \qquad (1)$$

is assumed constant between any pair or orthogonals, and by use of this relationship the change in wave height may be determined. It is obvious that diverging orthogonals, corresponding to submarine canyons, are associated with wave height attenuation. Conversely, converging orthogonals, corresponding to submarine ridges, are associated with wave height amplification. It may be noted that where favorable bottom topography fortuitously occurs at a chosen operational site, a certain amount of "breakwater" action will be obtained from the natural wave refraction phenomena. However, the large scale of topographic irregularity necessary for appreciable wave height attenuation precludes the use of artificially created topographic conditions as a mobile breakwater.

Another case of wave refraction occurs when waves advance into a region where a current exists. In this case, the wave velocity with respect to a fixed frame of reference is equal to the vector sum of the wave velocity with respect to the water and of the water velocity (current) with respect to the fixed frame of reference. The analysis of this problem follows the same method as that for the case of velocity change due to changing water depth.

For the case of an opposing current and deep-water waves, an analytic expression for the change in wave steepness can be obtained. This result, as shown in Fig.1, shows that an opposing current of velocity one-fourth the wave velocity will cause a deep-water wave of any initial steepness to build up a steepness of 1/7 and so presumably break and dissipate its energy (Scripps Institution of Oceanography, 1944). Such a current could be considered a mobile breakwater.

It may be pointed out that whereas a current of 10 to 12 ft/sec. - which might be considered barely feasible of attainment - would provide effective breakwater action, this is only true for deep-water waves, and it is unlikely that breakwater protection would be desired so far offshore as to insure deep-water wave conditions.

A more complete analysis, not restricted to deep-water waves yields a transcendental equation for the change in wave length (Carr, 1950):

$$\frac{L}{L_o} = \frac{\sqrt{gd}}{C_o} \sqrt{\frac{\tanh \frac{2\pi d}{L}}{\frac{2\pi d}{L}}} + \frac{V}{C_o} \qquad (2)$$

where the subscript 0 refers to initial conditions outside the current zone and V is the current velocity.

This equation can be solved by trial, Fig.2, for given initial conditions and the wave length L and group velocity Cg of the refracted waves so determined. The change in wave steepness can then be computed:

$$\frac{H}{L}\Big/\frac{H_o}{L_o} = \sqrt{\frac{C_{g_o}}{C_g}} \ \frac{L_o}{L} \qquad (3)$$

It can be shown that in this case a current of any velocity will only cause waves to reach a steepness ratio of 1/7 if the initial steepness is greater than some minimum. However, for any initial conditions, there always exists a critical current velocity for which no waves can traverse the current zone. From basic energy considerations, this current value is:

$$V = -C_g \qquad (4)$$

and this result can also be derived from Eq.2.

As a numerical example, it may be pointed out that the current velocity required to prevent transmission of 10-sec. waves in a water depth of 50 feet is about 25 ft/sec., and such a high velocity current is surely impractical.

WAVE INTERFERENCE

A progressive wave train traveling in the x-positive direction may be represented by the equation for the surface elevation at any point x and time t:

$$y_1 = \frac{H}{2} \sin 2\pi \left(\frac{t}{T} - \frac{x}{L} \right) \tag{5}$$

If a second wave train of identical period and amplitude, but with phase difference ϕ travels in the same direction, its surface elevation is:

$$y_2 = \frac{H}{2} \sin \left[2\pi \left(\frac{t}{T} - \frac{x}{L} \right) + \phi \right] \tag{6}$$

and the net motion becomes:

$$y = y_1 + y_2 = \frac{H}{2} \sqrt{2(1 + \cos\phi)} \sin \left[2\pi \left(\frac{t}{T} - \frac{x}{L} \right) + \alpha \right] \tag{7}$$

Thus, the resultant wave height will vary from twice the original down to zero as the phase difference varies from zero to 180° (one-half wave length).

This principle of wave interference could be applied as a mobile breakwater if some means could be devised for producing the secondary wave.

Interference can also be considered as occurring when waves advance into a region characterized by a vertically stratified current. Thus, if a current exists in a surface layer, the wave motion in this layer will be advanced or retarded with respect to the wave motion in the undisturbed deeper regions, and destructive interference can occur. It has been shown by G.I. Taylor (1943) that this mechanism is responsible for the (limited) performance of the pneumatic breakwater, a device often proposed as a mobile breakwater. Because of its prominence in the literature, this device will be described in some detail at this point.

Pneumatic Breakwater

The pneumatic breakwater, as conceived by Philip Brasher and patented by him in 1907, 1921, and 1929, consists of a submerged pipeline containing spaced discharge holes and supplied with compressed air from a ship or the shore. The resulting screen of rising air bubbles is claimed to prevent the passage of incident wave trains.

Although at no time did the inventor advance any sound analytic basis for the claimed performance, he was successful in obtaining several
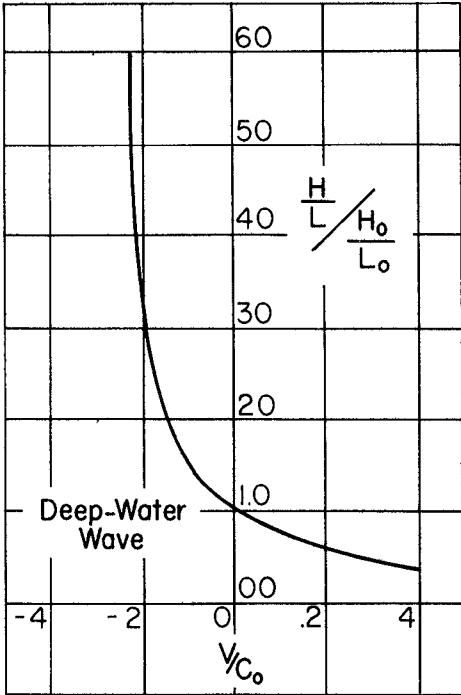
Fig. 1. Change in deep-water wave steepness due to refraction by a uniform current of velocity V.
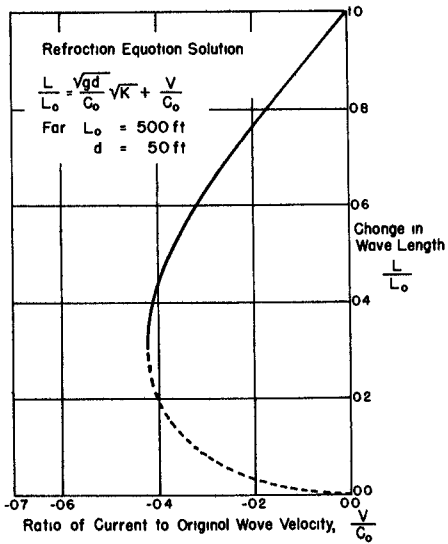
Fig. 2. Graphical representation of the general equation for wave refraction by a uniform current.
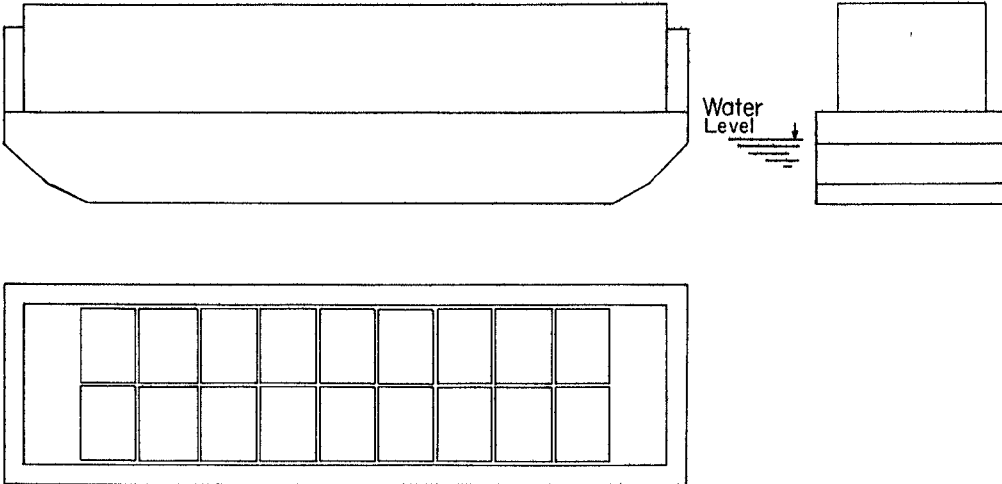
Fig. 3. Diagrammatic sketch of the "Phoenix" breakwater.
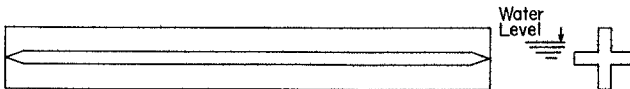
Fig. 4. Diagrammatic sketch of the "Bombardon" floating breakwater.

full-scale trials of his device in the period 1907 - 1929. One of the most extensive trials was at the Standard Oil Co. pier at El Segundo, California; but the results on this occasion, as on the others are not clear, there being a wide disagreement between inventor and client as to the value demonstrated. At any rate, at no time was a trial installation made permanent.

When plans were first being made for the invasion of Europe, the pneumatic breakwater was one breakwater device considered. Model studies performed by the British in 1942 were the basis of the first careful analysis of the mechanics of the pneumatic breakwater and showed the severe limitations of its usefulness. These experiments proved that any wave suppressing action of the air breakwater is due to the upward water current induced by the rising bubbles, and that duplicate results could be obtained when the water current is produced by any means, such as jets or propellors.

In 1943, G.I. Taylor made a complete analysis of the problem and derived a relationship between the length of wave which can be damped and the magnitude of the horizontal surface current resulting from the vertical current induced by the bubble screen. Calculations based on Taylor's theory show that the current required to damp very short period waves to be moderate and conceivably practical of generation by means of an air bubble screen, but for wave lengths or periods of the order to be expected in typical coastal environments the current values and corresponding power requirements to generate these currents become enormous.

WAVE DISSIPATION

Wave energy can be dissipated in the form of heat through the mechanism of fluid turbulence. For the dissipation to proceed at a high level, the turbulence must be general and violent; this is the wave breaking process.

Wave breaking occurs naturally on shelving coasts, where most wind-generated wave energy is finally dissipated. The process of wave breaking on such shorelines is preceeded by the increase of wave steepness (due to shoaling) to the point of instability.

It is obvious that artificial offshore bars or reefs for the purpose of inducing wave breaking are a very limited form of mobile or artificial breakwater. Another possibility for inducing wave breaking is a submerged vertical barrier that comes close to the still water level. It has been shown that such barriers can induce wave breaking under some conditions (Morison, 1949) (where the wave is already near the condition of instability). However, such a barrier is also a fairly efficient reflector and as such must be designed to withstand rather large forces.

WAVE REFLECTION

A fixed vertical barrier projecting above the water surface produces 100% reflection of incident waves. Thus, for normal incidence, the amplitude of the reflected wave train, $Hr/2$, is equal to the amplitude of the incident wave train, $Hi/2$, and at the barrier the variation of the water level with respect to time becomes $Hi \sin 2\pi t/T$.

An important consideration in the reflection process is the question of the magnitude of the forces acting on the reflecting barrier. These forces can be computed in terms of the wave heights by use of several formulas, such as that due to Sainflou, but in the present case a simpler scheme will be used.

For shallow-water wave conditions, which may be expected to be nearly the case for the water depths in which breakwaters are to be used, the pressure distribution below the surface is very nearly hydrostatic, hence the force on a barrier with mean water depth d and with the assumption of sinusoidal waves becomes:

$$F = \tfrac{1}{2} w \left(d + Hi \sin 2\pi \tfrac{t}{T} \right)^2 - \tfrac{1}{2} w d^2 \tag{8}$$

or, neglecting the H_i^2 term,

$$F = w d\, Hi \sin 2\pi \tfrac{t}{T} \tag{9}$$

For a <u>submerged</u> vertical barrier, the wave system includes the incident and reflected waves:

$$\frac{Hi}{2} \sin 2\pi \left(\tfrac{t}{T} - \tfrac{x}{L} \right)$$

$$\frac{Hr}{2} \sin 2\pi \left(\tfrac{t}{T} + \tfrac{x}{L} \right)$$

and the transmitted wave:

$$\frac{Ht}{2} \sin 2\pi \left(\tfrac{t}{T} - \tfrac{x}{L} \right)$$

The phase relationships between these waves at the plane of the barrier can be determined by use of the requirements of conservation of energy:

$$H_i^2 = H_r^2 + H_t^2 \tag{10}$$

and continuity, which requires the horizontal particle velocity to be
continuous across the plane of the barrier. On this basis, the differ-
ential water surface elevation across the plane of the barrier is:

$$H_r \sin 2\pi\left(\frac{t}{T} + \alpha\right)$$

Thus, if a submerged barrier is of sufficient height to produce
appreciable wave reflection, it will be subjected to horizontal forces
comparable in magnitude to those computed for the preceeding case.

APPLICATIONS OF WAVE REFLECTION

There are in general two methods by which the forces acting on a
mobile reflecting barrier could be resisted. In the first case, the
barrier could be designed as a gravity dam, with sufficient mass and
base width to prevent overturning, and in the second case the barrier
could be restrained by a mooring system, with anchors or pilings as the
resisting elements. It is interesting that both of these design ap-
proaches were developed in full size units for the Normandy invasion of
World War II. The following brief description and history of these
developments gives some idea of what is required of a mobile breakwater
in the form of a fixed reflecting barrier.

PHOENIX

The principal wave protection at the Normandy invasion harbors was
afforded by breakwaters constructed of block ships and special reinforced
concrete caisson structures called "Phoenix" (Wood, 1948). The design of
the Phoenix units was governed by the following set of conditions:
  (1) Maximum tidal range 22 feet.
  (2) Theoretical trochoidal waves 8 feet high and 120 feet wave
      length.
  (3) Minimum freeboard when sunk, 6 feet.
  (4) To be capable of being towed at $4\frac{1}{2}$ knots in a force 4 wind
      by a 1000 horsepower tug.

The resultant design was an open-top cellular caisson, with one lon-
gitudinal dividing wall and ten cross-walls, as shown in Fig.3. The units
were all approximately 204 feet in length and varied in depth from 35 to
60 feet and in width from 44 to 62 feet, the variation in the latter di-
mensions being in accordance with the water depth in which the individual
units were to be sunk. The design included scow-shaped ends to achieve
the required towing condition.

Construction of 147 of these units, sufficient for six miles of
breakwater, was completed in 150 days, utilizing several dozen building
sites and a labor force in excess of 20,000. The units were for the most

part built in graving docks, although twenty-four of the smaller units were built on ways and side-launched.

The units were successfully towed to the two invasion harbor sites, St. Laurent (American) and Arromanches (British), the first units arriving on D + 1. At Arromanches, 15 blockships and 7 Phoenix units were in place by D + 4, an additional 12 Phoenix units by D + 9, and 6 more by D + 12. During this same period, substantial numbers of caissons were also successfully placed at the American harbor. The time required to position a caisson and complete its sinking was of the order of 90 minutes, of which about 16 minutes was required for the actual flooding and sinking process.

The breakwaters formed by the rows of Phoenix units completely fulfilled expectations until the storm of June 19 - 22, 1944. This storm, which produced wave heights up to 12 feet and lengths up to 350 feet off the invasion coast, resulted in virtually complete destruction of the breakwater at St. Laurent. At Arromanches, an out-lying reef partially protected the artificial harbor and only five of the caisson units were destroyed. As a result of the storm damage, the St. Laurent site was abandoned and further harbor development work concentrated at Arromanches. By D + 44, 38 additional caissons were installed at Arromanches, 20 more by D + 71, 16 more by D + 119, and 14 more by D + 149. Most of units installed subsequent to D + 44 were for the purpose of repairing damage and preparing for winter storms by placing caisson units to seaward of the original line of block ships.

Failure in the original Phoenix design was due to two causes; excessive internal pressure when overtopped by large waves and excessive hogging moments induced by scour of the sea bed material at the scow-shaped ends of the caissons. The design of the caissons used in the later stages of the harbor construction was modified to include a complete reinforced concrete deck, which both prevented excessive flooding of the interior and provided much greater resistance to hogging. Many of these improved units were still sound several years after the War, and were raised and towed to Sweden for use in the construction of an oil dock in Stockholm harbor.

BOMBARDON

The design and construction of Phoenix was primarily the responsibility of the Royal Engineers. A parallel, and in a sense competing program, "Bombardon" was carried out at the same time by the Royal Navy. (Lockner, Faher and Penney, 1948).

Bombardon was designed as a floating vertical wave reflector, projecting far enough below the surface to intercept most of the incident wave energy and remaining fixed in position by a combination of its

dynamic characteristics and a mooring system. The units were constructed of 1/4 inch steel plate in 200-foot long units, each unit weighing approximately 250 tons. In section the units were cross-shaped with approximate depth and beam of 25 feet and draft of 19 feet, as shown in Fig.4. The horizontal arm of the cross was submerged so the beam at the water line was but 5 feet.

The cross-section was so designed to combine large mass (including the mass of water compelled to move with the hull) and small restoring force (due to the small waterline cross-section), and thus have rolling, pitching, and heaving periods long compared with the expected wave periods. By this means it was expected that the units would execute only small vertical and rolling motions when exposed to wave conditions. The units were designed to be moored in line with seaward and leeward anchors with a gap of 50 feet between the units. The units were connected to each other with twin 18-inch manila rope strops. In order to further reduce the wave energy which would be transmitted through the gaps, it was planned to use two parallel rows with staggered gaps.

Test sections of such a breakwater were constructed and installed in Weymouth Bay by early April 1944. These trials proved the units to be very successful for the design conditions, typical results being the reduction to 2 feet height of waves estimated to be 8 feet high and 170 feet wave length, incident on the breakwater.

One mile of Bombardon breakwater, installed in a single line, were installed at both invasion harbors by D + 6. The breakwaters were observed to perform as expected, reducing wave heights in their lee by about 50%, which is equivalent to an energy reduction of 75%. The Bombardon units withstood the first 30 hours of the storm of June 19 - 23, but eventually were completely destroyed. If may be noted that not only were the mooring stresses imposed by the storm waves greatly in excess of the design conditions, but that the longer periods of the storm waves approached the resonant periods of rolling and heaving of the floating structures, thus producing large amplitude motions in these modes.

REFLECTING BARRIER WITH MOTION

The reflecting barriers previously considered are designed to be fixed in space, and must therefore develop reactions sufficient to oppose the wave pressures developed by the reflection process. An interesting theoretical possibility is a barrier which is permitted some horizontal oscillating motion. Such a barrier cannot develop total wave reflection, but can produce appreciable reflection, hence permit tolerably small wave transmission. An important corollary of the barrier's motion is that the required mooring force may be appreciably reduced.

The limiting case for such a barrier is that of no restraint, in which case the motion of the barrier of weight W is described by the equation of dynamics:

$$\frac{W}{g}\ddot{x} = F(t) \tag{11}$$

where

$$F(t) = wd(y_1 - y_2) \tag{12}$$

and

$$y_1 = \frac{H_i}{2}\sin 2\pi\frac{t}{T} + \frac{H_r}{2}\sin\left(2\pi\frac{t}{T} + \alpha\right)$$

$$y_2 = \frac{H_t}{2}\sin\left(2\pi\frac{t}{T} - \beta\right)$$

w = specific weight of seawater.

y_1 = water surface elevation measured from still water surface at seaward face of barrier.

y_2 = as y_1 , at leeward face of barrier.

H_i, H_r, H_t = incident, reflected and transmitted wave heights.

α, β = arbitrary phase angles to be determined from the continuity requirements.

The solution of this equation gives:

Transmission coefficient,
$$C_t = \frac{H_t}{H_i} = \frac{1}{\sqrt{1 + \left(\frac{\pi W}{wLd}\right)^2}} \tag{13}$$

Reflection coefficient,
$$C_r = \frac{H_r}{H_i} = \frac{1}{\sqrt{1 + \left(\frac{wLd}{\pi W}\right)^2}} \tag{14}$$

The relationship between transmission coefficient, C_t and the parameter W/wLd, which is the ratio of barrier weight per unit width to weight of sea water per wave length per unit width is plotted in Fig.5.

With the provision for some restraint in the form of an elastic mooring of spring constant K, the dynamic equation becomes:

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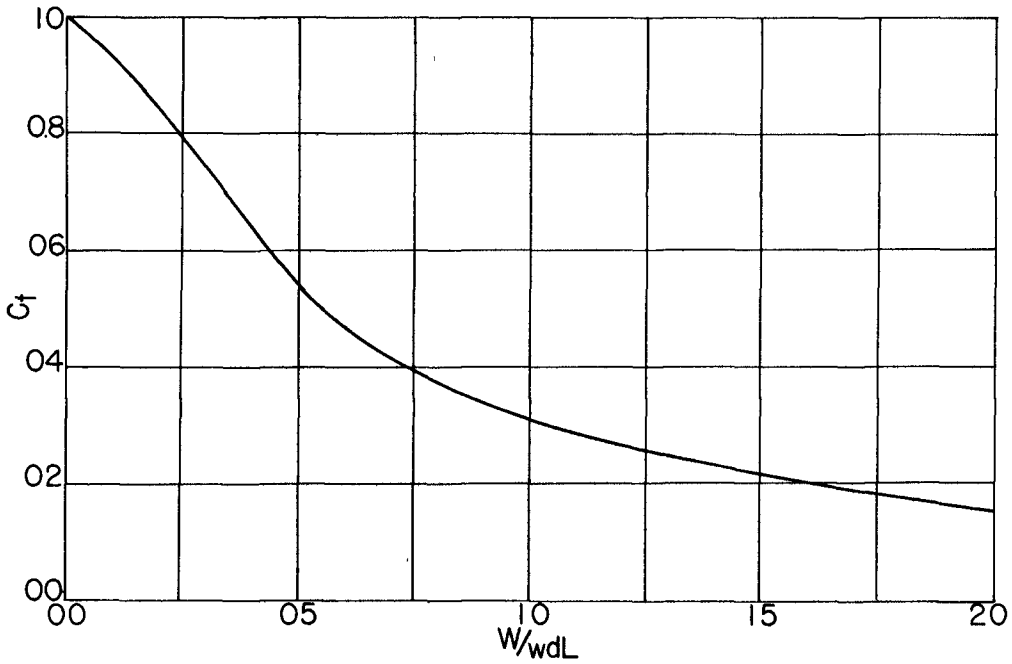
Fig. 5.
Theoretical transmission coefficient as a function of W/wLd for a freely floating (no horizontal restraint) barrier.
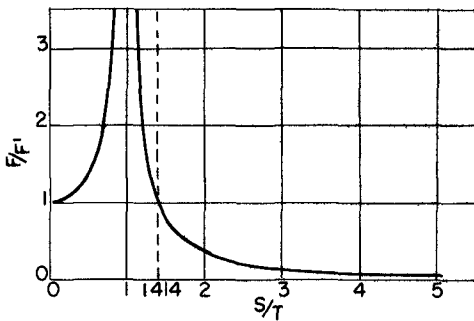
Fig. 6.
Ratio of maximum mooring forces for elastically and rigidly moored barriers as a function of S/T.
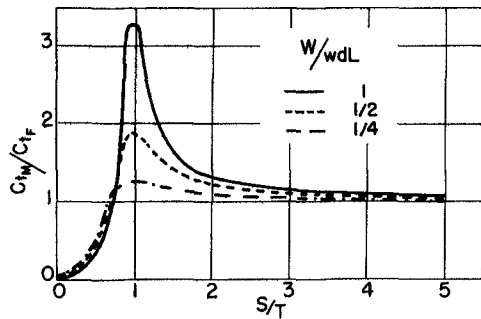
Fig. 7.
Ratio of transmission coefficients for elastically and freely moored barriers as a function of S/T.

$$\frac{W}{g}\ddot{x} + Kx = F(t) \qquad (15)$$

and $$C_t = \frac{1}{\sqrt{1 + \left(\frac{\pi W}{wLd}\right)^2 \left(\left(\frac{S}{T}\right)^2 - 1\right)^2}} \qquad (16)$$

where $S =$ natural period of barrier mass - mooring spring system

$$= \frac{2\pi}{\sqrt{\frac{Kg}{W}}}$$

The maximum force per foot of breakwater length becomes:

$$F_{MAX} = \frac{wd\,Hr}{1 - \left(\frac{S}{T}\right)^2} \qquad (17)$$

It may be noted that the corresponding maximum force on a rigidly fixed barrier is

$$F'_{MAX} = wd\,Hr$$

hence $$\frac{F_{MAX}}{F'_{MAX}} = \left|\frac{1}{1 - \left(\frac{S}{T}\right)^2}\right| \qquad (18)$$

This relationship is plotted in Fig.6.

The effectiveness of the partially restrained barrier may be conveniently expressed as a ratio to that of the completely free barrier:

$$\frac{C_{t\ MOORED}}{C_{t\ FREE}} = \sqrt{\frac{1 + \left(\frac{\pi W}{wLd}\right)^2}{1 + \left(\frac{\pi W}{wLd}\right)^2 \left(\frac{1}{\left(\frac{S}{T}\right)^2} - 1\right)^2}} \qquad (19)$$

This relationship is plotted in Fig.7 for several values of the parameter W/wLd.

From the plots of Figures 6 and 7 it is seen that for values of $S/_T$ greater than about 2, the mooring force is greatly reduced from that required of a fixed barrier, while the coefficient of transmission is increased very little over that of a completely free barrier.

FUTURE DEVELOPMENTS

It is anticipated that further development of mobile breakwaters will make use of wave reflection as the principal basis of operation. It also seems clear that the problem most in need of solution is that of providing resistance to the wave pressures inherent in the reflection process.

If mobile reflecting barriers may be divided into two broad classes, floating and fixed, it is further suggested that floating barriers, as typified by Bombardon, deserve more intensive development. The advantages of a floating system over a gravity system are many, chief among which may be listed:
     (1) Economy of material.
     (2) Freedom from foundation problems.
     (3) Freedom from erosion problems.
     (4) Indifference to tidal changes in site water depth.
     (5) Probable relative ease of transportation.

In the future development of moored floating barriers, two specific objectives may be listed:

     (1) Hydrodynamic design of the hull to obtain very long
         natural periods of rolling, pitching, and heaving.
     (2) Design of mooring systems, particularly the anchor
         points, to obtain greater ultimate strengths.

Any development program on floating barrier should also investigate more thoroughly the practical possibilities of free or elastically-restrained barriers of large mass.

REFERENCES

Carr, J. H. (1950). Mobile Breakwater Studies:
    Hydrodynamics Laboratory Report No. N-64.2,
    California Institute of Technology, Pasadena, California.

Johnson, J. W. and O'Brien, M. P. (1946). Graphical Construction
    of Wave Refraction Diagrams: Technical Report HE-116-221,
    University of California, Berkeley, California. (Unpublished)

Lochner, R., Faher, O., and Penney, W. (1948). The Bombardon
    Floating Breakwater: The Civil Engineer in War, Vol. II,
    Institute of Civil Engineers, London.

Morison, J. R. (1949). Model Study of Wave Action on
    Underwater Barriers: Report No. HE-116-304,
    University of California, Berkeley, California. (Unpublished)

Scripps Institution of Oceanography (1944). On Wave Heights in
    Straits or Sounds where Incoming Waves Meet a Strong Tidal Current:
    Wave Report No.11, La Jolla, California. (Unpublished)

Taylor, G. I. (1943). Note on the Possibility of Stopping Sea Waves by
    Means of a Curtain of Bubbles: British Admiralty Report ATR/MISC 1259.

Wood, C.J.R. (1948). Phoenix:
    The Civil Engineer in War, Vol.II
    Institute of Civil Engineers, London.

Chapter 26

STRUCTURAL ASPECTS OF LIGHTHOUSE DESIGN

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INTRODUCTION

The purpose of this paper is to provide an acquaintance with light-houses, the types of structures which have been erected, the reasons for their shape and form, and the basis for design. It will be revealed as the treatise proceeds, that the technique of design of light-houses has improved with the increase in knowledge of the forces that are acting and the behavior of materials while subjected to stresses and strains of these forces.

The subject under consideration is termed "pharology" which is defined as the scientific theory and treatment of signal lights and lighthouse construction. The paper confines itself to a concise discussion on lighthouse construction, covering briefly structures of historical importance to some contemporary designs. The concluding portion of this treatise deals with a proposed replacement of a light-ship by a fixed structure.

The development of lighthouse construction has been evolutionary. Thousands of years ago an aid-to-navigation consisted of a massive tower, atop of which a fire was burned. The exuding smoke aided the mariners in sighting landfall by day and the glow of the fire served the same purpose at night. The last century and one half has seen rapid strides in the improvement of lighthouse techniques. Progressively, the tallow candle replaced the open fire which in turn was supplanted by the oil and wick lamp and so on until today lamps and lenses have been developed to produce a beam of light with the magnitude of intensity of 25 million candlepower.

HISTORICAL

The Pharos of Alexandria is one of the Seven Wonders of the Ancient World (Fig. 1). Pharos, from which the term "pharology" is derived, was the name of the islet upon which the lighthouse structure was erected. The island was a long, narrow strip of land protruding in front of the ancient city of Alexandria, protecting the harbor from wind storms and high tides of the Mediterranean Sea. It was claimed that the light could be seen for 35 miles. A structure erected at sea level would have to be about 550 feet high to be seen at such a distance. If the description of the edifice is to be believed, then the Pharos of Alexandria is the most remarkable lighthouse structure of all time.

Fig. 1
The Pharos of Alexandria

WINSTANLEY'S LIGHTHOUSE AT THE EDDYSTONE.

Fig. 2

Fig. 3
Smeaton's lighthouse at Eddystone, England.

Fig. 4
Rare print of the old Boston lighthouse.

Erected in the second century B.C., it withstood the vicissitudes of time and the elements until destroyed by an earthquake in the 13th Century. Historian Edrisi, 600 years ago wrote: "This pharos has not its like in the world for skill of construction or for solidity; since, to say nothing of the fact that it is built of excellent stone of the kind called Redan, the layers of these stones are united by molten lead and the joints are so adherent the whole is indissoluble, though the waves of the sea from the north incessantly beat against it. From the ground to the middle galley the measurement is exactly seventy fathoms and from this galley to the summit twenty-six." This measurement would credit the edifice with a height of 576 feet.

The Romans, in the expansion of their empire, carried on the construction of lighthouses and while virtually all of them have fallen into decay, their existence is confirmed through coins and medallions unearthed by archeologists.

Skipping the Dark Ages of Western Europe, the next endeavors of historical significance were the British efforts at Eddystone in the English Channel. Eddystone is a reef of rocks lying in deep water fourteen miles southwest of Plymouth Harbor. The lighthouse is notable for many reasons. It is the anchor light in the chain of lights which enables safe navigation close to the English shore and it also marks the entrance to Plymouth Harbor. At high water the rocks are barely visible; at low water, the rocks appear as a low ridge of land. The existing structure is the fourth replacement, and the accounts of the design and construction of the first three structures make very interesting reading. For instance, Winstanley, the builder of the first tower, erected an ornamental structure consisting of an iron-legged tower with an architectural clothing of granite, and was convinced of its invincibility to the sea (Fig. 2). He expressed the wish to be in the tower during the greatest storm known in the English Channel. His wish was fulfilled about a year after completion of the structure when the tower gave way to the onslaught of the sea. Six lives were lost in the tragedy. The second structure was erected on a timber grillage with alternate courses of oak timber and granite. It was totally lost in a fire which started in the lantern housing. No sketch is available of the design. The third attempt was made by John Smeaton, an instrument maker (Fig. 3). It was of solid granite construction and marks a milestone in the annals of construction history in that he was the first designer to use natural cement as the binder in masonry construction. Smeaton's structure was replaced after 125 years of service. It might still be in existence were it not that the foundation partially gave way, rendering the superstructure unstable. The committee evaluating Smeaton's handiwork expressed the opinion that the tower itself was good for another century of use but that the difficult problem of underpinning the foundation and the need for a higher tower warranted the erection of a new structure.

The pattern for the lighthouse was set. Only a massive granite structure resting on a firm foundation can withstand the forces of the wind and the sea. Solid foundations immediately break up or deflect the heavy seas, and the spray alone rises up to the height of the lantern gallery.

EARLY AMERICAN LIGHTHOUSES

The first lighthouse established in this country was the Boston Lighthouse located on Little Brewster Island in Boston Harbor in 1716 (Fig. 4). Its construction was authorized by the Massachusetts Assembly. Initially the maintenance of the light was paid for by a tonnage tax, but the formation of the Federal Government brought the colonial lights, toll free, under a single jurisdiction. At the first session of Congress, an act was passed and subsequently approved in 1789 creating the Lighthouse Service which was later placed under the Department of Commerce. The Reorganization Act of 1939 made the Lighthouse Service a part of the Coast Guard under the Treasury Department.

In 162 years of activity, the Lighthouse Service and its successor, the Coast Guard, has built many major aids-to-navigation. Today, there are over 400 major structures in active service operated by the Coast Guard through its several district offices. Each of these structures has a story to tell and the romance of several has been compiled and published (U. S. Coast Guard, 1951).

Advancements made in the design of lighthouses and the techniques of construction, while not spectacular, have been notable. Each site offers some new challenges not previously experienced. Superficially, lighthouses appear to have a set pattern, but a perusal of the files of drawings reveals wide variations in foundation requirements for structures which are otherwise similar. The mariners' needs dictate heights for the lantern housing, and geography and the elements determine the materials of construction. There are no standard drawings or typical structures for the primary lights. You will discern from the accompanying illustrations that not all lighthouses are massive, high, conically shaped masonry towers, atop of which sets a lantern housing. The design of lighthouse structures has kept pace with architectural and engineering trends of the times.

Lighthouses may be classified as to site location which forms a criterion for design. They are as follows:

1. Spray-swept Structures - located on shore or on promontories.
2. Wave-swept Structures - located at water's edge or in the water.
3. Structures resisting ice pressures - located on inland waterways and lakes.

Fig. 6
Iron-legged lighthouse at Minot's ledge, outer Boston Harbor, Mass. Erected 1850, destroyed 16 months later.

Fig. 5
Spectacle Reef lighthouse, Lake Huron.

The nature and intensity of external forces in a large measure dictate the materials of construction. The forces expected to be encountered also depend on the geographic location of the site. Super-structures are made of the following materials:

1. Granite
2. Brick
3. Cast-iron plate towers
4. Skeleton steel structures
5. Reinforced concrete

One aspect of the type of foundation to be used is dictated by the depth of water which is encountered. Water depths may be classi-fied as follows:

0 to 10 feet	shallow depths
10 to 20 feet	moderate depths
20 feet and over	deep water

The superstructure in combination with the subsoil conditions, of course, governs the type of foundation to be used. Foundations may be classified as follows:

1. Screw pile foundations (helicoidal screws of cast-iron or steel attached to the piles)

2. Gravity foundations
    a. Pile supported
    b. Rock near surface
    c. Sand foundations
        (1) Open end caisson
        (2) Pneumatic caisson
        (3) Interlocking steel pile structure
        (4) Crib supported
        (5) Grillage and cylinder

Wave swept structures are located on sites where wave action must be taken into consideration both as to the stability of the structure and the difficulties encountered in construction. Two general types have been successfully used: (a) Solid masonry towers wherein the mass of masonry counteracts the external forces created by wave and wind actions and (b) open skeleton towers so designed that the struc-ture offers the least possible resistance to the kinetic energy produced by the waves.

Some of the most famous lighthouse towers have been built of solid masonry construction (Fig. 5). After repeated losses the lesson brought to bear was that the relentless pounding of the sea could only be with-stood by structures having solid masonry bases with all joints dove-tailed, circular in cross section, and having the exterior surfaces free from obstructions.

Fig. 8   Minot's ledge lighthouse.
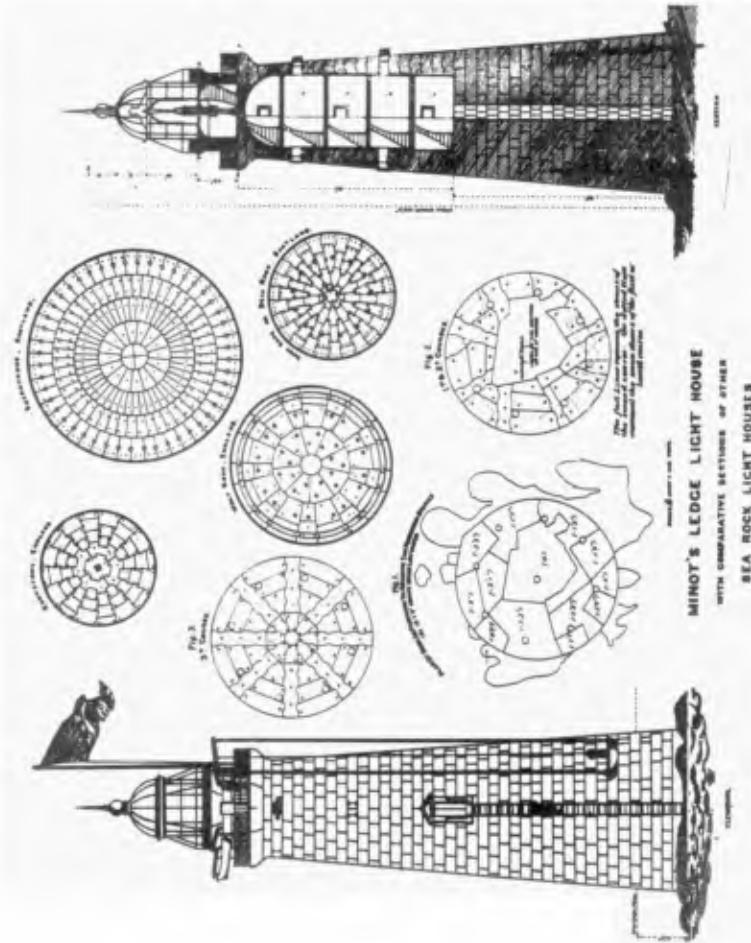
Fig. 7

A case in point is the Minots Ledge Lighthouse, the sentinal located one mile offshore and 6 miles southeasterly of Boston Harbor. In 1850, a 75-ft. iron frame structure was completed (Fig. 6). It lasted only 16 months before being swept to sea by a furious storm in which two of the assistant light-keepers lost their lives. The principal keeper survived to relate the terrifying experiences of the ordeal. The rock at the site is exposed only at low water and then only for a short period of time during spring tides. The frame structure consisted of 9 ten-inch wrought iron pipes, grouted into holes drilled five feet into the rock. Eight of these pilings were symmetrically spaced on the circumference of a 25-foot diameter circle with the ninth piling at the center. This operation took the greater part of two seasons to complete. The piling converged to the circumference of a 14-foot circular cast iron cap, 38 feet above the base on which the keeper's quarters were erected. The top of the lantern housing was 75 feet above the base. Two sets of cross bracing were provided for the posts but the lower set was never installed. The center of gravity was high, the lighthouse was fully exposed to the sea, and during intense storms the waves overtopped the lantern housing. Basically, the structure was inadequate, made particularly so by omission of the lower level crossbracings. The structure failed at the intended juncture of the crossbracing.

The present structure, started in 1855 and completed five years later, ranks as one of the great sea-rock lighthouses of the world from the viewpoint of overcoming the engineering difficulties encountered and the skill and science used in construction (Figs. 7 and 8). The rock bed had to be cut to proper shape before any of the stones for the courses could be laid. Preparation of the foundation took three years to complete. The conical tower is thirty feet in diameter near the base and 115 feet high. The lower forty feet is solid except for a central well. The tower contains 1079 granite blocks each weighting approximately 2 tons. The stones were cut on the mainland and check assembled before shipment to the site. They are cut so that each stone vertically dovetails to the neighboring stones. The courses, two feet thick, are held together vertically by bonding bolts wedged in place. The tower is massive and excellently constructed as attested by almost a century of service. Keepers report that in exceptionally heavy seas the lashing waves cause the tower to tremble to its very foundation.

REEF LIGHTS OF THE FLORIDA COAST

The Florida Reefs have always been a menace to navigation because of the convex nature of the reefs with respect to the channel in the Straits of Florida. The Gulf Stream runs close by, and the reefs of coral extend offshore for several hundred yards in shallow water ranging in depth from three to eleven feet. The shelf then drops abruptly.

Fig. 10 Cape Hatteras lighthouse, North Carolina (constructed in 1870).

Fig. 9 Carysfort Reef light station, Carysfort Reef, Florida.

To give some idea of the treachery of this water, from 1831 to 1844 there was always work in the channel to keep 50 wrecker vessels busy. To alleviate this deplorable condition, a series of six lighthouses was constructed between 1852 and 1880 marking the Florida Reefs from Fowey Rocks to Sand Key. These sentinels of the sea are generally referred to as the Reef Lights, and are all similar in construction, being of the steel skeleton type. The structural material used was wrought iron which has successfully withstood the action of the elements with a minimum of expense for maintenance and replacement of brace rods.

Carysfort Lighthouse: The Carysfort Lighthouse which was completed in 1852 stands in three feet of water about three hundred yards from the edge of the reef on a surface stratum of hard coral over a softer material below (Fig. 9). It is one hundred and seventeen feet high above the reef. The two-story dwelling is thirty-three feet above the water, with a cylindrical inclosure above containing a winding stair leading to the lantern housing. The framework is pyramidal in shape with a fifty foot diameter base with eight wrought iron piles forming an octagon. An additional pile is located in the center. The stratum of coral was deemed to have insufficient bearing power to support the piles, and large cast iron discs or foot plates, six feet in diameter, were used to spread the load over the coral. The discs have a hole in the center through which the piles pass. The piles were driven into the coral until a collar attached to the piles rested on the discs.

Sand Key Lighthouse: Sand Key is a low deposit of sand on a coral reef. In violent storms or hurricanes, the key is submerged and the surface sand is shifted. Shortly after subsidence of the storm, the key reappears in its original position but in a slightly modified form. Sand Key Lighthouse is similar to the Carysfort Lighthouse except that it is supported on piles screwed into the earth. The piles are of wrought iron, 8-inches in diameter, having attached at their tip ends cast-iron helicoidal screws with flanges two feet in diameter. The piles and screws total 13 feet in length and were slowly bored into the sand to a depth of ten feet below low water. Nineteen piles were installed in this manner. In addition to the screws, the 12 exterior piles pass through heavy cast-iron discs four feet in diameter which rest on beds of concrete. The Officer-in-Charge of Construction was Lieutenant George G. Meade who ten years later was in command of the Union forces at Gettysburg. Three years after completion of the structure, the island was washed away in a gale. In 1865, a hurricane again took away the island and everything on it except the lighthouse, which speaks well for the screw type anchor foundations. Today, after nearly a century of service, the framework is in good condition. Work is underway to protect the structure from corrosion by means of cathodic protection.

Sombrero Key Lighthouse: The Sombrero Key Lighthouse is the tallest of the Key lights and was designed by Lt. Meade after his experience in constructing the Sand Key structure. It is fifty six feet in diameter and one hundred and sixty feet high, supported on nine wrought iron piles twelve inches in diameter, resting in cast-iron discs eight feet in diameter. The lighthouse was built in six feet of water.

LIGHTS ON ISLANDS AND PROMONTORIES

The caprices of nature sometime fall in the category of the spectacular. There are numerous instances where the seas have swept away aids-to-navigation, both large and small. In the history of lighthouse construction, in the United States and in foreign countries, many primary lighthouse structures have succumbed to the relentless pounding of the sea and surf. However, there are two instances wherein engineering has successfully stayed the hand of doom. They are the lighthouses at Sand Island and at Cape Hatteras.

Sand Island Lighthouse: Sand Island Lighthouse is situated in the entrance to Mobile Bay, Alabama. The present tower, with granite trimmings, is the third to have been built, and was completed in 1873 on what was then a bank of sand, 400 acres in extent. At the time of construction, it was acknowledged that sands of the island were shifting, but a more suitable location nearby was not available. The present structure, 132 feet high, resting on a foundation of 178 piles, was located on what was thought to be the most stable part of the island, but in 60 years the outer edge of the island has moved more than a half mile. The hurricane of 1906, which brought such disaster to the Gulf Coast, completely washed away the island. Since then, riprap in prodigious quantities has been laid to protect the structure from being undermined by scour. During the hurricane of 1916, the keeper reported that the vibration of the tower was so great as to displace half the water in a fire bucket in the watch tower.

Cape Hatteras Lighthouse: Cape Hatteras has experienced the erection of three towers. The first tower, 90 feet high and made of stone masonry was built in 1798. It was in use for over seventy years before being replaced by the second tower which was completed in 1870. The second tower, 193 feet above the ground, is the highest brick lighthouse in the world (Fig. 10). It was struck by lightning eight years after completion and subsequently large cracks appeared in the masonry walls. Remedial action was taken which saved the tower, but soon after its completion, there began a very gradual encroachment of the sea upon the beach. This erosion became serious when, in 1919, the high water line came within 300 feet of the base of the tower. Several attempts were made to arrest this erosion, by dikes and breakwaters, but these courses of action proved of no avail. In 1935, a

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light was established on a skeleton steel tower placed farther back
from the sea on a sand dune. The brick tower was then abandoned by
the Lighthouse Service and placed in the custody of the National Park
Service. The Works Progress Administration erected a series of wooden
revetments which checked the wash that was carrying away the beach to
the extent that in 1942, the Coast Guard resumed its control and estab-
lished a lookout station in the old tower which by this time was 500 to
900 feet inland from the sea, and again tenable as a site for a light.

DEEP WATER FOUNDATION LIGHTHOUSES

Fourteen Foot Bank Lighthouse: The Fourteen Foot Bank Lighthouse is
located in Delaware Bay (Fig. 11). Completed in 1887, it was a daring
project for the times because it was the first lighthouse constructed
in the United States using a pneumatic caisson process for placing the
foundation. Plans were modeled after the Rothersand Light Station,
Weser River, Germany. Both structures were built about the same time
and in the same depth of water.

This lighthouse is presented because it clearly illustrates the
method of constructing a pneumatic foundation. A timber working
chamber 40 feet square with sides seven feet high was built on shore
and launched. Over the crib were placed three courses of cast-iron
cylinders 35 feet in diameter, 6 feet high made of built-up plates
bolted together along the flanges (Fig. 12). An air shaft with air
lock, through which the men passed to the working chamber, was erected
on the foundation cylinder and braced to the shell plates. The portion
above the working chamber was filled with concrete to give the caisson
a draft of $15\frac{1}{2}$ feet. The caisson was towed to the site and was lower-
ed into position in about twenty feet of water, by admitting water in
the cylinder. The current produced considerable scour as soon as the
caisson was grounded and it continued to sink about 8 feet until the
roof of the working chamber rested on a mound of sand. Sufficient
air pressure was maintained in the working chamber to keep out the
water. The excavation was made in the caisson within the cutting
edges, the sand being blown out through a blow pipe within the air
shaft. The space within the shell was filled with concrete as the
work progressed and about 1000 tons of riprap was placed around the
base to prevent scour. The total height of the foundation cylinder
is 73 feet. After 65 years, an inspection of the station indicates
the structure is in excellent condition.

Many cylindrical structures of the type of architecture repre-
sented by the Fourteen Foot Bank Lighthouse have been built. However,
the type of foundation and consequently the method of construction
have been varied to suit the subsoil conditions encountered. In
shallow water, the cylinder is placed directly on the prepared bed
with riprap protection around the base. As deeper penetrations were
required to secure mass to resist external forces, the site was
dredged to achieve the desired depth. In deeper waters, the coffer-
dam and caisson methods have been adopted.
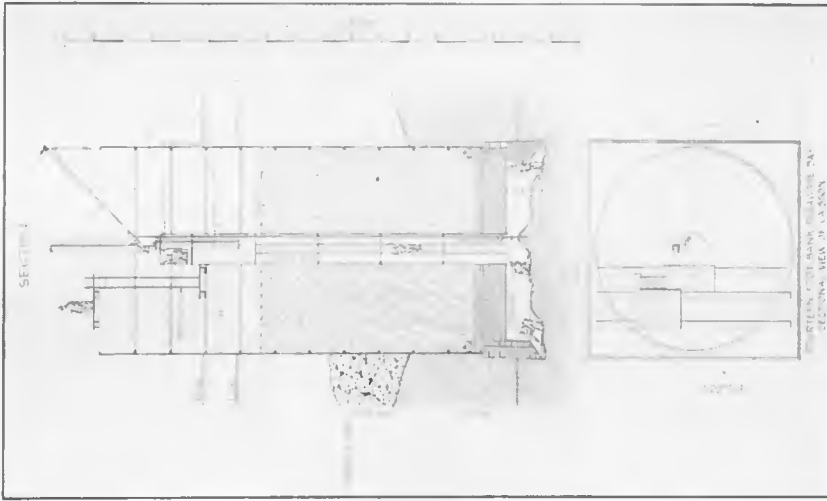
Fig. 12

Fig. 11
Fourteen Foot Bank lighthouse, Delaware Bay
(Established 1876, rebuilt 1886)

Cleveland Ledge Lighthouse: The Cleveland Ledge Lighthouse is one of the primary light structures most recently designed and built by the Coast Guard (Fig. 13a). It is located in Buzzards Bay, Massachusetts, at the western entrance to the Cleveland Ledge Channel approach to the famous Cape Cod Canal. The light station was commissioned on June 1, 1943, after three years of construction. The contract price for the project was a quarter million dollars, but the contractor defaulted after eight attempts to sink the foundation and the bonding company completed the job.

The structure is located in 21 feet of water on a sea bed of glacial drift consisting of sand, gravel and small boulders. The presence of boulders ruled out a sheet pile cofferdam type of design and a prefabricated caisson design was adopted instead. The steel caisson was erected on a timber grillage at a marine launchway. It was launched in the usual manner of launching a ship and then ballasted before being towed to the site and sunk.

The shell consisted of two concentric cylinders (Fig. 13b). The inner cylinder, which was assembled first, was 42 feet in diameter and 40 feet high. The outer cylinder, 52 feet in diameter and also 40 feet high, was erected around the small cylinder. Horizontal struts between the outer and inner shells of 2-inch pipe were spaced three feet on centers both horizontally and vertically. The two cylinders were assembled on a timber grillage of two layers of 12 by 12 inch full length timbers with a 1/2 inch space between each timber. The two layers were laid crosswise one to the other and drift pins at alternate crossings integrated the timbers into a unit. The shell was secured to the grillage by curved steel angles riveted to the outer and inner cylinders and bolted to timbers below.

A reinforced concrete mat 3 feet thick was then cast inside the inner cylinder over the timber grillage. The reinforcing bars of the mat were extended into the five foot annular space through holes drilled through the wall of the inner cylinder. Concrete was also cast in the annular space to a height of 8 feet. High-early-strength concrete was used throughout the job. The caisson was launched at this stage of construction.

Prior to towing to the site, reinforced concrete cross-walls, 3 feet thick and 18 feet high, were built within the inner cylinder, forming four quadrants. Each quadrant was then subdivided in half by an 18 inch thick wall and 10 feet high above the concrete mat. The reinforcing bars of the cross-walls extended into the 5-foot annular space, through holes drilled in the inner cylinder wall. Sufficient reinforced concrete was placed in the annular space to provide a draft of 19 feet. A temporary timber platform was then erected over the top of the structure and the caisson was towed to the site.

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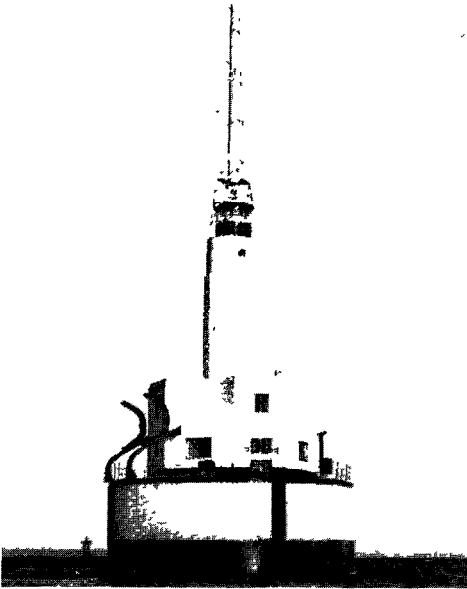
Fig. 13a

Fig. 13b

Cleveland Ledge lighthouse, Buzzards Bay, Mass.
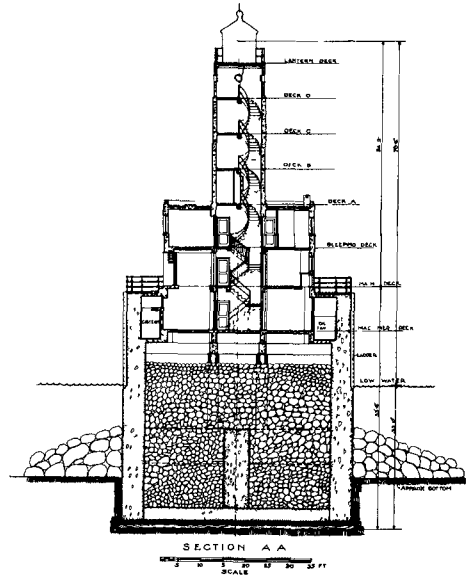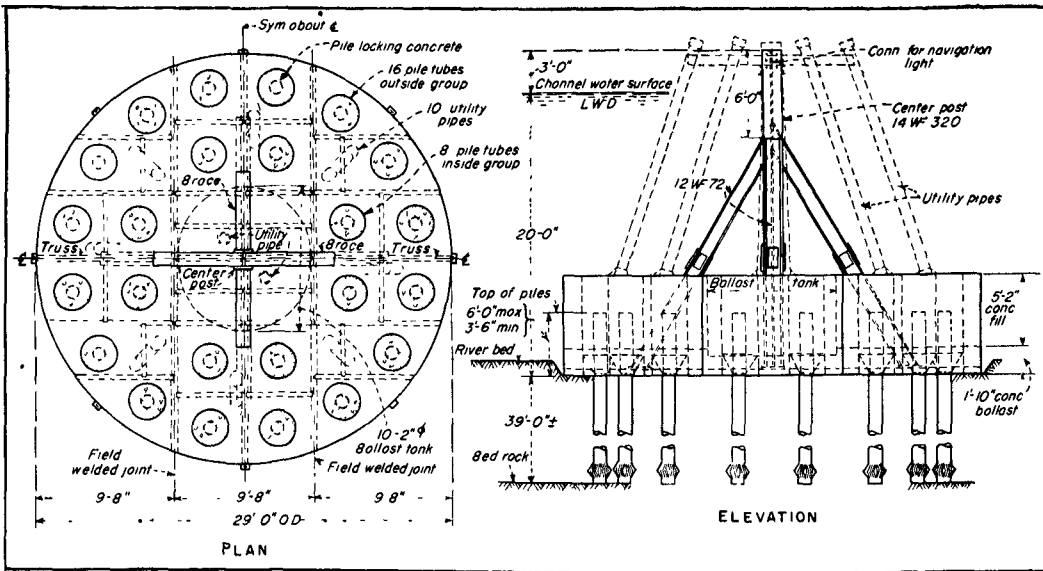
Fig. 14
The "Aquapad". Special expansion piles driven
through pile tubes anchor the structure.

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Meanwhile, the site was prepared by dredging a circular hole about 13 feet deep. The foundation was leveled off by divers and a one foot layer of crushed stone was spread over the bottom of the hole and leveled.

The caisson was lowered onto the prepared bed by opening flood valves and flooding the inner compartments and the annular space. The sinking operation was completed in $1\frac{1}{2}$ hours. However, the caisson did not come to rest properly on the prepared bed. The structure had to be refloated in order to redress the bed. This cycle of operations was repeated 9 times before the structure was acceptably seated. Prolonged foul weather, a listing of caisson while afloat and strong currents which made controlled sinking difficult proved too much for the contractor.

Sand fill then was dumped into the depression around the structure and leveled off by divers using a fire pump and hose. Riprap of stone from 1 to 3 tons each was placed around the structure to prevent scour.

The inner cylinder was ballasted with stones up to 10 inches in diameter and the compartments were de-watered to the top of the stones. The annular space was completely de-watered and filled with reinforced high-early-strength concrete. All subsequent operations were accomplished in the dry. The portion of the structure exposed to the sea from below low-water and above high tide is protected by a band of 3/8 inch thick wrought iron plate.

West Neebish Channel Light No. 13: The lighthouse structures so far described have been constructed either on land or in moderate depths of water. Primary lights in deep water are invariably installed on lightships. Lightships, of course, are not fixed structures and it is not unusual for them to drag anchor and be off position during severe storms. Then, again, lightships are unsuitable where heavy shifting ice or deep land-locked ice is encountered. For example, the yearly recurrent drama that makes newspaper headlines, is the great race which ore boats wage against ice floes on the Great Lakes. A premium is paid the boat and crew that succeeds in bringing the season's last shipment of ore from the Mesabi Range in Minnesota to the open ports in the lower lakes. These 600 foot goliaths must navigate the St. Mary's River via the West Neebish Channel which is only 300 feet wide. The channel is beacon-lighted quite similar to an airport runway, the lights being located at the edge of the channel where the water depth is 25 feet or more. Before the arrival of spring, Coast Guardsmen traverse the ice to prepare the lights for the ice-breakers which precede the first ore boats on their initial trip back to the range. It can be seen, therefore, that only fixed light structures are suitable for this waterway. They must be stable against both ice floes and stationary ice. Heretofore, the gravity type

structures depended on dead weight to resist the overturning and sliding forces imposed on them. The greater the assumed external forces, the more massive became the structure which in turn increased the surface presentment to the external forces of wind, ice and waves.

This paper so far has developed two basic concepts of design for lighthouses. Massive structures whose gravity loads are adequate, with a factor of safety, to overcome the external forces applied, and secondly, skeleton structures with relatively small applied forces and which are anchored in firm bottoms. The first mentioned type structure is exemplified by the Fourteen Foot Bank Lighthouse. In order to construct such structures, a large amount of equipment and large numbers of men working under hazardous conditions are necessary. The Cleveland Ledge Lighthouse required a cumbersome caisson which had to be floated into position. Subaqueous excavation and site preparation was necessary. The principle of using mass as the counteracting force is not very efficient because of Archimedes' law of buoyancy. There was also described the skeleton type lighthouses as exemplified by the Reef Lights of Florida wherein the surface presentment to the externally applied forces is comparatively small. The almost free standing piles which are characteristic of skeleton type structure would not be practical in deep waters especially where ice is encountered. It was found that the screw-tip type piles are subject to scour in the presence of ice floes. Many of the minor light structures supported on screw piles in the Delaware estuary had to be replaced for this reason.

The problem which presents itself, therefore, is to keep to the minimum the surface presentment to the external forces and to incorporate the desirable features of gravity type structures. The late Mr. John P. Emschwiller, formerly Chief Structural Engineer in Coast Guard Headquarters has solved the problem with an underwater foundation called the "Aquapad" (Anonymous, 1951). In essence, the aquapad is a prefabricated foundation which is towed to the site and sunk into position by ballasting with water. The aquapad is fixed to the sea bed by driving piles into pile tubes and interlocking the two structural elements with concrete or grout (Fig. 14). Anchorage for the integration are deformation rings both on the pile tubes and on the heads of the pile.

The principles of design of the aquapad can best be described by referring to the West Neebish Channel Light No. 13. The shell is made of 3/8 inch thick plate, 27 feet in diameter and 7 feet high. The proportions of the structure and the sizes of the elements are determined from the principles of ship design and are predicated on the depth to which the structure is to be sunk. Light No. 13 rests

in 24 feet of water. There are 24 pile-receiving tubes 30 inches in diameter which also serve as supplementary supports for the bottom plate and the top deck. The shell is further supported and integrated by internal trusses and circumferential angle iron bands and split Tee beams. The shell is watertight and contains a water ballast tank in the center. The heavy vertical post is the sole support of the superstructure, while braces at the quadrant positions laterally stay the center post and compel the structure to act as a unit when under stress. The feature especially to be noted is the small area of the center post which presents itself to ice action. For purposes of discussion, say that under the given circumstances, ice has a crushing strength of 800 lbs. per square inch and that the ice thickness is 20 inches. The center post, a 14 WF 320 lb. column core section, has a flange width of 16-3/4 inches with the major axis placed parallel to the direction of the prevailing ice movement. The center post accepts an ice thrust under these conditions of approximately 267 kips. Existing structures of the gravity type are about 20 feet in diameter which results in an ice presentment of about 4800 square inches. The ice thrust for gravity structures is approximately 4,000 kips. It becomes evident why almost all of the gravity type structures in the West Neebish channel have been replaced at least once, and practically all of them in use today have a definite list. The continued use of the gravity type structures entails a program of periodic replacement.

The Aquapad was designed to be fabricated in three sections and to be shipped by rail to an assembly and launching site. Shop fabrication included the installation of the stub sections of the center post and the four braces. Assembling of the three sections and installing the portions of the center post and braces above the deck was accomplished by field welding. The shell was launched and moored to a dock where the work of installing the reinforcing bars was completed. A predetermined amount of concrete ballast was poured in the bottom of the shell to give it a freeboard of $6\frac{1}{2}$ inches. The shell was lashed to the back of a scow and towed to the site. Despite inclement weather, the aquapad was lowered into position in $6\frac{1}{2}$ minutes by flooding the ballast tank. About half of the piles were driven in place and grouted in before the shell was de-watered. Driving the remainder of the piles completed construction.

Lake Huron Lightship Replacement: The aquapad installation at the West Neebish Light No. 13 has passed its initial winter's test and is considered a success. Coast Guard Headquarters has plans under way towards applying the aquapad principle to the replacement of the Lightship in Lake Huron at the entrance to Port Huron. The depth of water at the proposed site is 26 feet. The subsoil exploration reveals the strata below the lake bed to be sand.
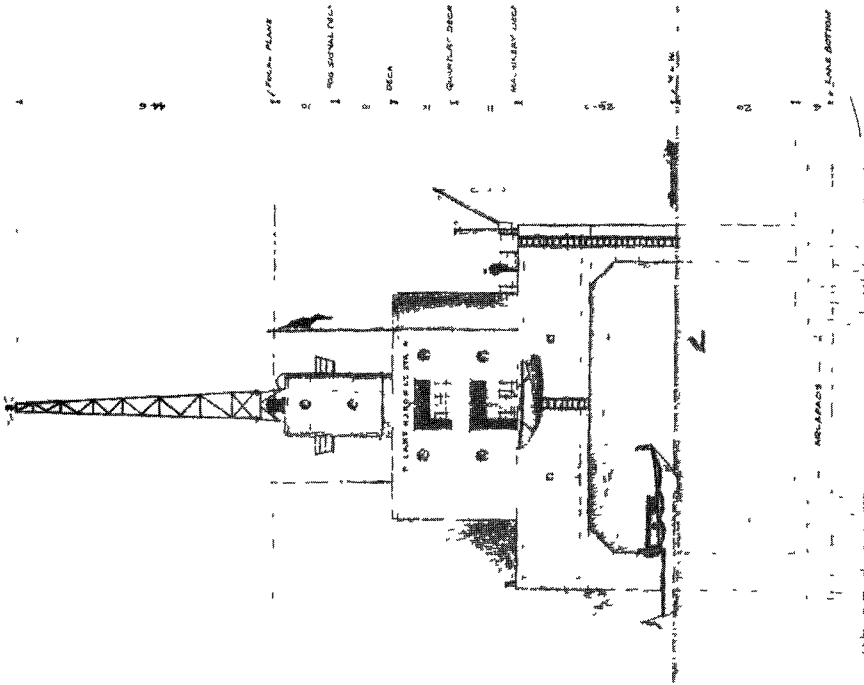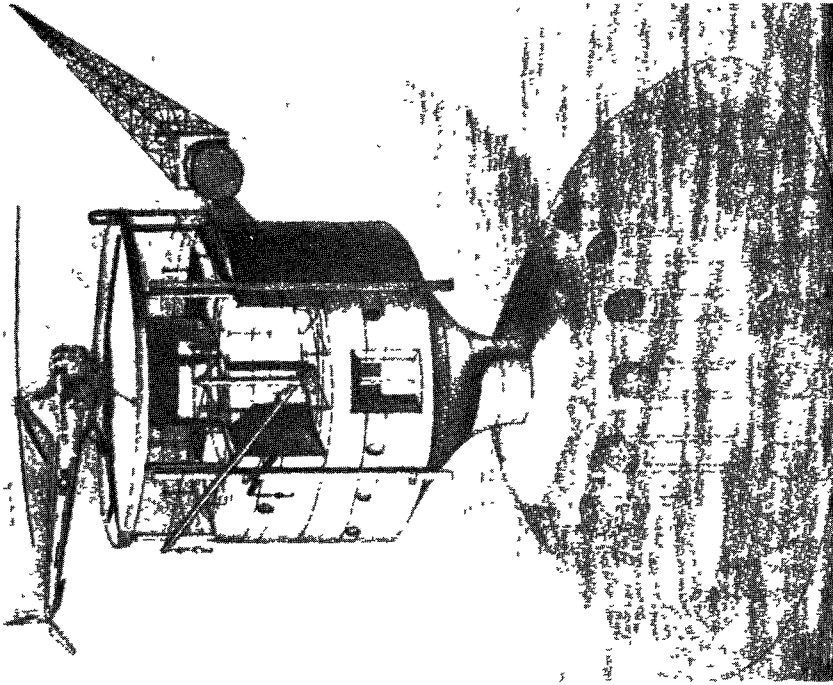
Fig. 16
Design concept of future aid-to-navigation.

Fig. 15
Lake Huron lightship replacement.

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A fixed structure is proposed as the replacement for the lightship on a foundation consisting of four aquapads. Four columns will emerge upright from the pads without interference of crossbracings at the water line and tied together at the top by haunched beams forming a three dimensional rigid frame (Fig. 15). Construction difficulties immediately present themselves. For instance, the question arises as to the practicability of setting the 4 aquapads sufficiently accurate in plan and elevation to subsequently accept the prefabricated superstructure. This requirement of construction is not too demanding when it is recalled that an aquapad is in effect a submarine and is easily maneuverable under water. In other words, with almost a nominal holding force, the aquapad can be either held in position or moved into any other position to make adjustments so necessary for prefabricated rigid frame construction in steel. The superstructure will be erected well above the high-water level. The lower deck will have a 5-foot working area around its periphery and will also be the machinery room floor. The upper deck will be quarters for the lighthouse attendants. Rising above the upper deck will be the tower structure supporting the lantern housing, with radio beacon antenna tower on top of the structure. The Lake Huron Lightship replacement will soon crystallize and in its successful completion will bring into full realization the significance of the aquapad principle of design in submarine construction.

The Coast Guard is projecting its design concepts even farther into the future. With the experience gained in successive applications of the aquapad, it is within the realm of practicability that lighthouses will someday have an architectural appearance of the nature shown in Figure 16. It is entirely functional and expressive of its structural system. Ice pile-ups will be of moderate significance since the spherical shape of the aquapad will minimize the effect of ice thrust against the structure. Further, the columnade of the superstructure will in some measure dissipate the energy of the waves. The radio antenna can be lowered by remote control to permit helicopter landings. It will be feasible to make helicopter landings when swells and rough water prevent tenders from approaching the lighthouse.

REFERENCES

Anonymous (1951). Here's a new type foundation for marine structures. Engineering News Record, vol. 147, pp. 41-43.

U. S. Coast Guard (1951). Historically Famous Lighthouses. Publication CG-232.

Chapter 27

PERTAINING TO PRESTRESS

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The purpose of this paper is to summarize in some detail the application of prestressed concrete to coastal structures, explain the basic theories of prestressed concrete, and give an explanation of an application of prestressed concrete recently designed and used by the author.

APPLICATION TO COASTAL STRUCTURES

Prestressed concrete can be easily adapted for use in coastal structures. One fine example that can be cited is the construction of offshore platforms used for drilling or stationary equipment. The platform itself could be constructed as a prestressed flat slab, stressed in two directions. The supporting piles could be constructed of precast, prestressed, concrete piles. At least one major pile company is now making hollow pile sections up to 36 inches in diameter which have proven to be very satisfactory. These piles are driven in the same manner as any other standard pile. Since the concrete section is always in compression, handling stresses are not critical. The concrete is never cracked, hence there is a very high resistance to salt water action. These piles also have the advantage of being fireproof, as well as having a low maintenance expense. A prestressed, precast, flat slab can then be constructed and placed on top of these piles. This slab will be much lighter in weight than an ordinary concrete slab and will also have the advantage of never cracking or spalling. Thus the salt water will not have any detrimental effects. This slab will also be highly resistant to fire, and maintenance costs would be low. The result of such construction will be a rigid, permanent structure, able to resist wind, impact, and waves, with a marked degree of success.

In the same vein of thought is the application of prestressed concrete to dock and wharve construction. Once again prestressed piles in conjunction with prestressed beams and girders would provide an excellent structural material for dock construction.

Standard warehouses, with span lengths greater than heretofore thought possible with concrete, could also be constructed from prestressed concrete. Columns, beams, girders, and floor slabs could all be prestressed. In many cases in Europe, the prestressed design of many buildings was chosen in competition over the reinforced concrete, steel, and timber designs. Recent advances in the design of prestressed concrete now enables the design to include continuous and rigid frame structures.

Another use for prestressed concrete is the construction of tanks and pipes. Watertightness is always of prime concern and by using pre-

stressing is virtually assured. Prestressing is applied in two directions in a method developed by the Preload Corporation. Both vertical and circumferential prestress is applied. This places the wall in compression a sufficient amount to overcome the tensile stresses caused by the outward pressure of the water. These tanks could be used for storage very satisfactorily. The same system is being used in the manufacture of concrete pipes. A 40 mile pressure pipe line, 30 inches in diameter with a two inch shell, designed to withstand pressures up to 280 psi, is under construction in Canada at present.

Barges have been built using reinforced concrete. They were unsatisfactory in that they were entirely too rigid and failed in tension when heavily loaded; however with prestressed concrete it would be possible to introduce enough precompression to overcome this failing and the end result would be a strong, water-tight barge. This method of construction would also be applicable to many other types of floating works including docks, breakwaters, floating platforms, and many other harbor structures.

ADVANTAGES OF PRESTRESSED CONCRETE FOR COASTAL STRUCTURES

There are many advantages to be gained in using prestressed concrete. The primary advantage is the great savings possible in the materials used in the construction. It can be estimated that a large quantity of steel will be saved along with a small amount of concrete. With present steel and wood shortages, there exists a need for a material which can replace these critical materials, but still be as good as the original material. Prestressed concrete has the full capability of filling this need. Prestressing also prevents any cracking of the concrete, thus making a very waterproof structure. There is no fear of marine borers or sea water action on prestressed concrete members as there would be on wood and steel members. Thus no added treatment is necessary nor is continued maintenance a large item of expense. Most of the structural members lend themselves easily to precasting or factory manufacture thus allowing a standardization of section which will decrease the initial first cost of the member. Long spans, heretofore not possible in reinforced concrete construction may be built giving large unobstructed areas for warehouses. The material has excellent fatigue properties and withstands impact loading very well, making it an excellent material for dock and pier construction. All of these advantages and still many others are being utilized by engineers in Europe and the United States in a great variety of structures.

FUNDAMENTALS OF PRESTRESSED CONCRETE

Concrete is inherently an excellent material in compression; however in tension it fails at a very small percentage of the load it can carry in compression. In flexural members, this inability to carry tensile forces is particularly detrimental. The steel placed on the tension side of the flexural member deforms a greater amount than the concrete, causing cracks which makes the tensile portion of the member

useless. This in turn decreases the shear resistance of the material thus causing large web sections to be introduced. This in turn adds to the dead weight of the member. In prestressed concrete these short-comings are remedied to a large extent.

Prestressing is nothing more or less than introducing stresses and deflections opposite in sense to those introduced when the structure is loaded with dead load and live load. These stresses are introduced into the members by tensioning high tensile strength wire which is placed as reinforcing within the concrete members. The tensioning can be done in two ways, either by pretensioning or by posttensioning the steel wire. Prestressed, posttensioned concrete indicates that the prestress has been introduced after the concrete is poured and set to a satisfactory strength. The ends of the member are used as reactions for a jack. In order to do this it is necessary to prevent bond from developing between the steel and concrete. This can be done in many ways including using an asphalt material, coring a hole with a rubber tube, or covering the rod with a paper or plastic material. When post-tensioned is used, the load is usually transmitted to the concrete by means of an end bearing device. In some instances grout is also forced around the wires after stressing and the wires bonded into the concrete. Prestressed, pretensioned, concrete indicates that the prestress has been introduced into the wire before the concrete is poured. When the concrete is set to strength the wire is released and the load is trans-mitted to the concrete by bond. There are many methods of prestressing, both pretensioning and posttensioning, used in Europe and the United States today, but the basic fundamentals are identical in each and every method.

As an example of prestressing theories, take a simply supported beam with a dead load of w#/ft and a superimposed live load of P#'s. The bending moments at the critical sections can be evaluated and from these moments both the dead load and live load stresses can be evaluated. The primary purpose of prestressing is to introduce prestress of such magnitude so that the dead load and live load tensile stresses are just balanced by the magnitude of precompression. As an example of this see Figure 2.

The stress diagrams in Figure 1 indicate when the beam is loaded with dead load and live load the bottom fibers are just at zero stress. This would indicate that the beam is now practically straight or similar to its condition before loading at all. Thus in the prestressed beam, the entire section is in compression making the entire depth effective in developing the internal resisting moment. Thus the ratio of depth of beam to span length is greatly reduced.

Shear stress when accompanied by diagonal tension is of prime importance in concrete design. In ordinary reinforced concrete shear stress is used as a measure of diagonal tension. It is necessary in the beam above, to provide reinforcing for the diagonal tension developed at the point of maximum shear. This reinforcing is provided by means of stirrups at the supports. In prestressed concrete however the con-dition is not the same. Shear stress is no longer taken as a measure

of diagonal tension and the values of the diagonal tension are also reduced by the action of the prestress forces. This can be shown quite effectively by the use of Mohr's circle of stress and the failure envelope that can be obtained for the concrete. A concrete cylinder indicated failure at 3000 psi when tested in compression. The same concrete failed in tension at a stress of 450 psi when subjected to a tensile loading condition. If the condition of stress is known on two mutually perpendicular planes, these conditions can be plotted on a circle of stress known as Mohr's circle. From this circle of stress the condition of stress existing on any plane can readily be obtained. With the results of tests known and shown above, two circles of stress can be drawn. From these two circles of stress the envelope of rupture can be drawn. Any condition of stress that falls outside of this envelope indicates failure of the material. The value of "C" indicates the shear strength of the material without any normal stress. Two cases of loading of the concrete are shown. Case I is the condition of stress that exists when ordinary reinforced concrete is used. If the value of the cohesive stress is developed, the member will fail in tension and therefore the full cohesive strength of the material cannot be developed. Consequently, it is necessary to add reinforcing and increase the thickness of the section. In case II with a prestressing stress of .3 of the ultimate strength of the concrete, the full cohesive strength of the material can be fully developed without failure of the member. Thus it is possible to decrease the section, which will decrease the shear stress due to dead load. This characteristic of prestressed concrete overcomes to a large degree one of the basic shortcomings of reinforced concrete.

Hence by using prestressed concrete the section behaves almost as a steel beam in that the section can be developed to its full depth to resist moment, the section remains in compression hence no tensile cracks are formed, and the resistance to shear is greatly increased.

PRESTRESSED ROOF SLAB

A prestressed slab, tensioned in two directions, was constructed at the Southwest Research Institute to determine the applicability of prestressing to flat slabs. The slab was posttensioned in two directions mutually perpendicular to each other. The slab was 88 feet in length and 38 feet in width. The thickness of the slab was six inches. The slab was supported on eight columns with a maximum span between columns of 24 feet and a maximum cantilever of 10 feet. The steel was laid and the slab formed, poured, and stressed on the ground and then lifted into place following the usual practices of the Youtz-Slick Lift Slab Method of building construction. In the Youtz-Slick Method, all slabs are poured on the ground, one on top of the other, and then lifted into place by hydraulic jacks placed on top of the columns. The slabs are lifted by long screws extending from the jack into the slab and fastened therein to a lifting collar. The lifting collar serves a dual purpose, both as a means of lifting, and as a structural shear head to distribute the reaction of the column into the slab. When the slab is lifted into place the slab is fixed to the columns by welding.

Fig. 2

MOHR'S CIRCLE OF STRESS

EQUATION OF MATERIAL

S = C + σ TAN φ
S = SHEARING RESISTANCE PER UNIT AREA
C = COHESION - SHEARING RESISTANCE PER UNIT AREA WHEN σ = 0
σ = COMPRESSIVE STRESS
φ = ANGLE OF SHEARING RESISTANCE

CASE 1
SHEAR STRESS = 600 PSI

ELEMENT

CASE 2
SHEAR STRESS = 800 PSI
PRESTRESS - COMPRESSION = 900 PSI

ELEMENT

Fig. 1

STRESS DISTRIBUTION IN CONCRETE

W - POUNDS PER SQ FOOT

BEAM LOADED WITH DEAD LOAD ONLY

DEAD LOAD STRESS

PRESTRESS

DEAD LOAD AND PRESTRESS

BEAM LOADED WITH DEAD LOAD AND LIVE LOAD

DEAD LOAD AND PRESTRESS

LIVE LOAD STRESS

DEAD AND LIVE LOAD AND PRESTRESS

f_c - COMPRESSIVE STRESS
f_t - TENSILE STRESS
f_{dl} - DEAD LOAD STRESS
f_{ll} - LIVE LOAD STRESS
f_{ps} - PRESTRESS STRESS

A one-sixth model of the roof slab was built in order to ascertain the behavior of the slab. All linear dimensions of the slab and steel were reduced by a factor of six. SR-4 strain gages and Ames dial gages were mounted at critical sections to determine stresses, deflections, and strains, in the slab when subjected to dead load and live load conditions.

DESIGN CONSIDERATIONS

The slab was analyzed and designed as a horiziontal gridwork of beams intersecting at right angles. Using this system the moments due to dead load and live load were determined and the section and quantity of steel necessary was obtained. The steel used was one-quarter of an inch in diameter, high tensile strength steel. The ends of the steel rods were upset and threaded through end bearing plates in groups of six and anchored at the ends of the slab. Since the steel was post-tensioned, a heavy coat of asphalt was applied to the steel to prevent bond. Poststressing was done in two different ways. Along the long dimension of the slab, each group was pulled individually using a stressing plate which remained in the slab after stressing. When the wires were fully stressed shims were placed between the stressing plate and the end bearing plate to hold the desired increment of strain. Across the short dimension, stress was introduced into the wires by separating sections of the slab. The slab was poured in two sections, with only the wires continuous across the opening. Openings were left in the slab for hydraulic jacks and when the concrete was set to strength the two sections of the slab were jacked open a predetermined distance. The opening was then grouted in with a fast setting grout and the jacks removed when the grout reached desired strength. The attempt of the slab to pull back together thus introduced the prestress into the slab. A working stress of 110,000 psi was developed in the steel and a stress of 1400 psi compression was developed in the con-crete. When the structure is fully loaded with dead load and live load, there is no bending tensile stress developed in the concrete.

The concrete mix was designed for a 28 day strength of 5000 psi. Cylinder tests indicated that this strength was reached in about 10 days. The concrete had a cement content of 7 sacks per yard. The concrete was poured with a slump of 3 inches.

MODEL TEST

The model test was performed to determine the behavior of the large slab. Readings of strain, load, and deflections were taken on the model and converted to full scale using the following relation-ships:

Item	Full	Model
Stresses (bending)	$S = \dfrac{Mc}{I}$	$S = \dfrac{Mc}{I}$
(shear)	$S_s = \dfrac{VQ}{It}$	$S_s = \dfrac{VQ}{It}$
Strains	$= \dfrac{S}{E}$	$= \dfrac{S}{E}$
Deflections	$= \dfrac{KWL^3}{EI}$	$= \dfrac{KWL^3}{6EI}$

These relationships indicate that stresses and strains are identical in the model and full size slab while the measured or evaluated deflections in the model are one-sixth of those obtained in the full scale test. This can be proven if it is assumed that the properties of the materials used in the model are identical with the properties of the materials used in the full sized slab. If a factor of 6 is used, the dimensional properties of section change in accordance with the power of the property. For example, the moment of inertia is reduced by a factor of $(6)^4$ since this property is $(in)^4$ and all linear dimensions are reduced by a factor of (6). The Total load W is reduced by a factor of $(6)^2$ because the cross sectional area is reduced by $(6)^2$. The unit load w, $\#/ft^2$ remains the same in the model and full slab however.

RESULTS OF MODEL STUDY

When loaded with a dead load and live load equal in magnitude to the design load of the full sized slab, the deflection readings indicated that the slab was substantially level. In other words the deflection due to the prestressing moments were just about balanced by the deflections due to live and dead load. From the study of the electric strain gage readings, it was determined that the concrete had not gone into tension in any of the measured portions of the slab.

Further loading indicated that deflections and stresses developed did not become abnormally large. The model is presently loaded with a uniformly distributed load of $212\#/ft^2$ or 2.25 times its designed load. There are no cracks or spalling and the deflections have remained in the same magnitude as before. The slab will be allowed to retain this load for a long period of time to determine any plastic flow or creep characteristics.

Another problem of particular concern was the shear developed around the lifting collar. The diagonal tension that is developed is one of the more critical items in the design of the slabs used with the Youtz-Slick Method. With a slab prestressed in two directions, the stress condition becomes triaxial. An analysis of this condition indicated that the magnitude of tension developed is very small compared with a conventional slab of the same dimensions. This analysis is very well substantiated by the model test as the magnitude of shear at the lifting collar is high enough to fail the concrete by diagonal tension under ordinary conditions.

GENERAL CONSIDERATIONS

In the slab constructed only 40 per cent of the weight of steel used in a comparable slab was used. There was only two-thirds of the usual amount of concrete used. Thus this indicates a substantial savings in dead weight which in turn lessens the size and cost of the footings, columns, and collars used. By using prestressed concrete, it is unnecessary to provide any built-up roof for waterproofing. With the use of prestress it will be possible to greatly increase the span lengths, and thus provide more unobstructed area.

CONCLUSION

There are many more possible uses for prestressed concrete. The material is applicable to many structures in all types of construction. It remains to be seen how American ingeniuty will adapt prestressing into standard construction procedures. The initial efforts have been made and have met with success. There are many disadvantages still to be overcome, but with the basic conceptions thoroughly established pre-stressed concrete stands on the threshold of nationwide acceptance.

REFERENCES

Freyssinet, E. - Prestressed Concrete - Principles and Applications; Institution of Civil Engineers, London, England.

Lalande, M. - The Variety of Applications of Prestressed Concrete; Cement and Concrete Association, London, England.

Magnel, G. - Prestressed Concrete - Its Principles and Applications; Cement and Concrete Association, London, England.

Magnel, G. - Prestressed Concrete - Concrete Publications Limited, London, England.

Parrett, J. T. - First Prestressed Piles Carry Tank Platform; Engineering News Record, July 5, 1951.

Chapter 28

SOME OCEANOGRAPHIC AND ENGINEERING CONSIDERATIONS
IN MARINE PIPE LINE CONSTRUCTION*

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ABSTRACT

The proper design of a pipe line for the transport of gas or oil from
sea to land requires the solution of a number of engineering problems
either not encountered in pipe line engineering on land or found to be of
a different nature in the marine environment than in the terrestrial
environment. These include: (1) consideration of the vertical stability
of the pipe, (2) consideration of the lateral stability of the pipe and
its vertical risers in the presence of wave-induced forces, and (3) con-
sideration of the longitudinal stability of the pipe in the presence of
thermally induced tensile and compressive forces. The first of these
considerations is treated in the present paper.

In those areas where the bearing capacity of the upper sediments is
small, as is the case for certain regions of the Gulf shelf, downward sag
of a pipe line can occur and entrenchment of the line to considerable depths
may be necessary in order that excessive stresses within the pipe be avoided.
Because both the flexural and longitudinal tensile stresses, occurring
simultaneously, can be important in a sagging pipe line, both must be
evaluated. Appropriate formulas and graphs are presented for this purpose.
From these and a knowledge of the sediment characteristics along the proposed
pipe line route, it is possible to determine whether or not regions of
critical sag might develop in a pipe of given specifications.

COASTAL ENGINEERING

INTRODUCTION

The present and potential source of offshore oil and gas in tideland regions of the Gulf of Mexico demands an economical mode of transportation from sea to land. Pipe lines can meet this demand, if designed not only to endure the processes of deterioration in the sea, but also to withstand the internal stresses induced by lack of adequate support, by severe wave loads, or by thermal changes.

An introduction to the scope of problems encountered in the design and installation of pipe lines to be laid upon or beneath long stretches of the sediments such as the continental shelf of the Gulf of Mexico has been given in another paper (Reid, 1951). The purpose here is to expand upon some of the physical problems which are encountered and to present, in summary, the results of theory and techniques which may be useful in the design of a marine pipe line.

Some of the specific questions which arise in connection with the laying of a pipe line offshore are:

(1)   What route should be followed in reaching a certain off-shore destination?

(2)   Can the pipe be laid upon the bottom or must it be buried within the sediments?

(3)   If the burial of the pipe line is indicated, what should be the depth of burial?

(4)   Will the pipe sink into the sediments; if so, how much sag will be experienced and what will be the stresses induced thereby?

(5)   Will support of the pipe in regions of weak sediment be required in order to insure vertical stability of the pipe, either from the standpoint of downward sag due to excessive net weight of pipe or from the standpoint of buckling associated with thermal expansion?

The engineers of the United Gas Pipe Line Company were confronted with problems of this nature in planning the 15 miles of 20.5 inch pipe and 10 miles of 14 inch pipe which has recently been laid within the sediments of the Atchafalaya Bay, Louisiana, and the adjacent Gulf. (A discussion of the preliminary investigation appears in the Petroleum Engineer, March 1951, and the installation of this line is discussed by Paul Reed, 1951.)

Such questions can be answered or at least partially answered by con-sidering the vertical stability of the pipe in the light of the general stratigraphy and strength distribution of the sediments along the path of the pipe line.

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VERTICAL STABILITY OF THE PIPE

GENERAL DISCUSSION

Adequate support of a pipe line resting upon or passing through the sediments of a marine environment, such as encountered on the Gulf Shelf, cannot be taken for granted. The conditions of sediment strength and degree of consolidation are considerably different from those which are encountered in the case of ordinary soils. According to the soil mechanics classification given by Terzaghi and Peck (1948), those soils having an unconfined compression strength of greater than 8,000 pounds per square foot are considered extremely stiff and those soils having a strength of less than 500 pounds per square foot are considered very soft. The different degrees of stiffness which make up the classification are contained between these extremes. In comparison, the mean unconfined compressive strength of silty clays and clayey silts encountered in the upper strata of the sediments of the Atchafalaya Bay and adjoining Gulf region, for example, has been found to be approximately 80 pounds per square foot.* Values range from less than 10 to about 250 pounds per square foot. All of these values fall in the very soft category. From the standpoint of pipe line engineering, it appears necessary to refine the classification since the relatively stronger portion of the very soft sediments can adequately support certain pipe lines. As an arbitrary limit those sediments having an unconfined compressive strength of less than 100 pounds per square foot (or a shear strength of less than 50 pounds per square foot) will be referred to hereafter as extremely soft. The extremely soft and very soft silty clays are of recent origin and increase in thickness (from a few feet to about 15 feet) with distance from shore in the Atchafalaya Bay area, forming a wedge of weak deposits resting on top of relatively stronger, and more consolidated, marsh deposits of considerable thickness. Even the latter deposits are soft in terms of the above classification.

This is a greatly oversimplified picture of the stratigraphy. Superimposed on this structure are "pockets" of nearly fluid sediment which apparently extend to depths as great as 10 or 15 feet. These pockets lie principally between regions of hard reef, and consequently represent a situation to be considered with caution because of the possibility of differential sag of the pipe. A route which passes through such zones may demand entrenchment of the pipe to considerable depth in order to avoid the possibility of overstressing in the pipe walls due to sag. Whether or not such sag could be critical depends upon such factors as the net weight and length of the section subject to deformation, the initial tension in the pipe, the strength of the sediments adjacent to the weak zone, and the depth of the weak zone.

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*   Leipper, et. al. (1951). Oceanographic Analysis of Marine Pipe Line
     Problems. Final Report to the United Gas Pipe Line Company.

The sag of a pipe section, having a length of the order of 200 feet or more, introduces a complex problem from the standpoint of computation of induced stresses. One is dealing here with a beam which is so long that when vertical deformation occurs it is accompanied by a significant elongation. The firmer sediment adjacent to the weak zone will tend to restrain the movement of the pipe at the ends of the sagging portion of the pipe so that practically all of the elongation will occur in the sagging section. This can induce a net axial tension of considerable magnitude. The tensile stress thereby induced in the material is in addition to the tensile and compressive flexural stresses induced by the bending of the pipe.

In the case of a very long pipe the bending effect can become so small that the sagging pipe can be considered essentially as a flexible cable. In this case the pipe will assume the shape of a catenary under the action of a uniform load per unit length, with the tensile force carrying the full load. If the pipe section is very short or if the deflection is very small, then the theory of simple bending may apply. In this case the net tension would be negligible and the load is carried entirely by shear forces. The situation regarding pipe sag in the sediments in general, involves both bending stresses and net tension, and the load is carried partially by shear and partially by tension. In order to insure a safe design where sag is likely to occur, it is therefore necessary to compute both flexural and pure tensile stresses induced by the sag.

CRITERION FOR SINKING OF THE PIPE

An offshore pipe line which is resting upon the bottom will exert a downward load on the underlying sediment which is simply the submerged weight of the pipe in water, or absolute weight minus the weight of water displaced by the pipe. In order that static equilibrium exist, the sediment must develop an equal and opposite reaction. There is a maximum reaction which the sediment can exert. This may be referred to as the ultimate load bearing capacity of the sediment. In general the bearing capacity depends not only upon the nature of the sediment but is a function of the applied load distribution as well. Thus a pipe will have an effect on the sediments which differs from that which a flat plate of the same weight would induce. The criterion for sinking of the pipe is that the net downward gravitational load exerted by the pipe is greater than the ultimate load bearing capacity of the sediment.

If the pipe is entrenched within the sediment, the problem of evaluating the net downward gravitational load of the pipe becomes somewhat complex. Evidently the load exerted by the entrenched pipe depends upon the structural nature of the sediment itself. In contrast to a suspension (which represents a dispersion of discrete particles in a fluid), the sediment consists of a continuous network of solid materials

which includes water within the interstices of the structure. The
solid phase presumably supports its own weight and does not add to the
hydrostatic pressure of the water phase as in the case of a suspension.
In a fluid sediment, neglecting capillary forces, there will be a
buoyant force exerted upon the pipe which will be equal to the weight
of water displaced by the pipe. However, since only part of the total
volume of sediment is water, the buoyancy will be less than that ex-
perienced by a pipe submerged in water alone. The water content of a
sediment is generally evaluated in terms of the per cent of the dry
mass of sediment. This will be denoted by the symbol Q . The
buoyancy per unit volume of the pipe in the sediment, however, is equal
to the mass of water per unit volume of the sediment. If B repre-
sents the buoyant force per unit volume of the pipe, then

(1)
$$B = \frac{\rho_s g}{1 + 100/Q} ,$$

where ρ_s represents the density of the sediment (i.e., the wet density).
As an example, consider a sediment having a specific gravity of 1.4 and
a moisture content of 100 per cent of the dry weight. In this case
$\rho_s g =$ 87.4 pounds per cubic foot, which leads to a value of B of
43.7 pounds per cubic foot. This represents a buoyant force which is about
70 per cent of that which would be experienced by a pipe submerged in
water alone.

If equilibrium is to exist, the sediment must support a greater
percentage of the actual weight of the pipe than in the case of a pipe
resting upon the bottom. The maximum reaction which the sediment can
develop with respect to the pipe will in general depend upon the
adhesive property of the sediment with respect to the pipe, the pressure
existing at the depth of entrenchment, the shear strength of the sediment,
the cohesive property of the sediment, and the size of the pipe. The
combined effect of bearing reaction at the bottom of the pipe and
adhesion along the sides and top of the pipe, under conditions of
maximum restraint, represents the ultimate load bearing capacity in this
case.

For silty clay sediments, the ultimate load bearing capacity is
evidently independent of the pressure within the sediment, and depends
only upon the shear strength and load distribution. If P_b represents
the ultimate load bearing capacity per unit length of pipe, D the
overall diameter of the protected pipe, and τ_u the ultimate shear
strength * of the sediment, then presumably

(2)
$$P_b = k_b D \tau_u ,$$

---

* This can be measured directly for a sample of sediment or can be
taken as one-half of the ultimate unconfined compressive strength
for clayey sediment.

for a silty clay. For the case of a flat strip load of width D acting on a flat surface of clay soil, Terzaghi (1943) gives 5.14 for the factor of proportionality k_b . In the case of a pipe, k_b is probably much smaller than this, judging from the limited information available on sinking of pipes. The circular shape of the pipe evidently leads to a stress concentration in the sediments beneath the pipe which is greater than that experienced in the case of a flat plate. For concrete coated pipe entrenched in silty clay sediment, an approximate value of $k_b = 2$ is indicated from the experience gained in the installation of United's pipe line.

It must be emphasized, however, that further information is needed for establishing the empirical validity of equation (2) as well as evaluating the proportionality factor. At the present time it is not possible to state the exact threshold of equilibrium existing for a pipe loaded sediment. It can be stated, however, that there is a significant probability that sinking will occur if the net load per unit length, exerted by the pipe, exceeds $2D\tau_u$, and little chance that it will not sink if the load exceeds $5D\tau_u$. If sinking is to be avoided the net load should be less than $2D\tau_u$.

SYMMETRICAL SAG OF A PIPE LINE IN PLASTICALLY DEFORMED SEDIMENT

The problem of determining the combined flexural and tensile stresses induced in the case of differential sinking of the pipe line is examined in this section. Two simple end conditions are considered for the sagging section of pipe. In the section which follows, an analysis of the end conditions for a relatively firm (but non rigid) supporting material is made by taking into account the elastic deformation of this material.

If the vertical restraint per unit length, P_m , offered by the plastically deformed sediment is uniformly weak in the zone of pipe sag, and if the conditions of support at the ends of the sagging section are similar, then the vertical deformation of the pipe will be symmetrical with respect to the center of sag.

The net vertical load on the pipe per unit length, w , is simply the net weight of the pipe in the sediment minus the reaction P_m . If w_p represents the weight of pipe per unit length in air (including the weight of transported fluid-gas or petroleum), then

$$(3) \qquad w = w_p - \frac{\pi}{4} D^2 B - P_m ,$$

where B is the buoyancy as already defined. The reaction, P_m , of the distorted sediment is not necessarily the same as the bearing capacity, P_b , of the undisturbed sediment.

The criterion for sinking, however, is that

$$\left(w_p - \frac{\pi}{4} D^2 B \right) > P_b .$$

Thus, if the net weight is great enough, then the sediment deformation exceeds the elastic state and its structure is broken down, reducing the possible reaction which it can develop.

The Theoretical Model

A model is envisaged in which a pipe line spans a pocket of excessively weak sediment of horizontal distance ℓ_o along the pipe, resulting in a net downward force w on the pipe over the length ℓ_o. The weak material is homogeneous from the standpoint of maximum restraint, and is of sufficient depth that the point of maximum sag of the pipe does not reach a layer of strong sediment below. Furthermore, conditions of elastic flexural deformation and elongation are presumed, such that the amount of sag is very small compared to ℓ_o and hence the slope of the sagging section is very much less than unity. The assumptions regarding the loading of the pipe and the elastic theory are summarized below:

(1) The net downward force per unit length, w, acting on the pipe is uniform along the pipe and essentially normal to the pipe.

(2) The downward force is independent of the vertical deformation of the pipe.

(3) The end conditions are the same at each end of the sagging section, such that the sag is symmetrical with respect to the point of maximum sag.

(4) The tension due to the axial elongation of the pipe is uniform throughout the entire length of the sagging section of the pipe.

(5) Plane transverse sections of the pipe remain plane after combined bending and extension of the pipe.

(6) The modulus of elasticity in tension is the same as that in compression for the pipe material.

(7) The proportional elastic limit of the pipe material is not exceeded.

(8) The axis of the pipe is initially straight.

(9) The slope of the sagging pipe is so small that the rate of change of the slope per unit length of pipe represents the curvature of the pipe.

(10) The pipe is of uniform cross section.

The assumption of negligible tension cannot be made for a sagging pipe line, as is done in the case of a simple beam, because of the magni-

tude of the deflection involved. If the maximum deflection in a fixed end beam is of the order of magnitude or greater than the width of the beam, then the restoring moment associated with the induced tension becomes appreciable. * In order to keep the complexity of the problem at a minimum, the assumption (4) is made. This appears reasonable provided that the tension is large compared with the limited amount of longitudinal restraint provided by the sediments in the weak zone.

Probably the most severe restrictions regarding the application of the theory are (1), (2) and (3), and to a less extent (4). In applying the theory to a real situation one must keep these assumptions in mind. Examples illustrating the use of the theory are given at the end of Part I, and certain modifications in the application of the theory are discussed.

The Basic Equations of Combined Flexure and Elongation in a Pipe

A schematic diagram of the sagging section of pipe is shown in Figure 1B, and the equilibrium of forces and moments is represented graphically by Figure 1C. The origin of the coordinate system is taken at the point of maximum sag; x represents the horizontal distance measured positively to the right of this point, and y is the vertical distance measured positively upward from this point. This allows a convenient form for the equations governing the deflection, in view of the fact that symmetrical sag is considered. The bending moment at the origin is denoted by M_o, and the total axial tension after deformation of the pipe is represented by N. An initial tension may exist in the pipe line, due to thermal or pressure effects within the pipe. This is denoted by N_o, and is represented graphically in Figure 1A. The shear at the center of sag is zero for symmetrical sag. In view of assumptions (1) to (10), the equation representing the balance of moments about point A in Figure 1C is

$$(4) \qquad M = EI \frac{d^2 y}{dx^2} = M_o - \frac{w x^2}{2} + N y,$$

where M represents the bending moment within the pipe at section A of distance x from the origin. The quantity E is the modulus of elasticity in tension (or compression) of the pipe material, and I

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* Usually the assumption (7) would be transcended in a short beam before the restriction involved here would govern; this however depends upon the flexibility of the beam.

Fig. 2. Schematic diagram of restricted sag, where a portion of sagging section is supported by firmer sediment at depth h_0 below original level of pipe. Equivalent free sag for a portion of the pipe illustrated in diagram B.

Fig. 1. Schematic diagrams of symmetrical, free sag in a pipe: (A) initial state, (B) deformed state, (C) balance of forces and moments in a section of the pipe, (D) illustration of fixed end condition, (E) illustration of condition of ends free to turn.

is the area moment of inertia of the cross-section. The latter can be expressed as follows:

$$(5) \qquad I = A_s r^2 ,$$

where A_s is the cross-sectional area of the steel in the pipe, and r is the radius of gyration of the cross-section of steel, taken about the neutral axis.

The balance of vertical forces between O and A is given by

$$(6) \qquad V = \frac{dM}{dx} = -wx + N\frac{dy}{dx}$$

where V is the shear force at section A. Consequently the shear at the end of the sagging section is

$$(7) \qquad V_1 = -\tfrac{1}{2}wl_0 + N\theta_1 ,$$

where θ_1 is the end slope at $x = l_0/2$ (the negative of that at $x = -l_0/2$). Because of the small values to which the quantity θ is restricted, it represents therefore the angle (in radians) between the pipe and the horizontal plane. It will be noted from this that the shear forces at each end carry only part of the total net weight of the sagging pipe, unless the ends are held rigid and the angle θ is zero. The total net weight wl_0, of course, must ultimately be sustained by the vertical reaction of the supporting material at each end of the sagging section.

The General Solution of the Equations for Symmetrical Sag

The solution of (4) for the vertical deflection of the pipe is

$$(8) \qquad y = \frac{1}{N}\left\{ (M_0 - w\lambda^2)\left[\left(\cosh\frac{x}{\lambda}\right) - 1\right] + \tfrac{1}{2}wx^2 \right\} ,$$

where λ is a characteristic length defined by

$$(9) \qquad \lambda = \sqrt{\frac{EI}{N}} .$$

The conditions $y = 0$ and $dy/dx = 0$ at $x = 0$ are employed in arriving at equation (8).

The expressions for the slope, bending moment and shear at any point in the sagging pipe can be derived from equation (8) as follows:

(10)
$$\theta \equiv \frac{dy}{dx} = \frac{1}{N}\left\{\frac{1}{\lambda}(M_0 - w\lambda^2)\left(\sinh \frac{x}{\lambda}\right) + wx\right\},$$

(11)
$$M = (M_0 - w\lambda^2)\left(\cosh \frac{x}{\lambda}\right) + w\lambda^2,$$

and

(12)
$$V = \frac{1}{\lambda}(M_0 - w\lambda^2)\sinh \frac{x}{\lambda}.$$

It can be shown furthermore that in the limit (8) reduces to

(8a)
$$y = \frac{1}{EI}\left(M_0 \frac{x^2}{2} - w\frac{x^4}{24}\right),$$

when N is extremely small. This equation, representing a special case of the more general relation (8), is that which the simple theory of flexure yields.

On the other hand, if N is very large then V is small and (8) reduces to

(8b)
$$y = \frac{wx^2}{2N},$$

which is the approximate form of catenary sag associated with tension N. These two limiting cases serve as checks on the more general theory.

Application of Hooke's Law for Evaluation of the Tension

Since the tension is one of the sought variables of the problem, an additional equation involving N is necessary in order to make the solution unique. This can be established by applying Hooke's law to the over-all extension of the sagging pipe section. If ℓ represents the length of the pipe section between points (1') and (1) after vertical

deformation has occurred, and ℓ_i is the initial length between the same points in the pipe prior to deformation, then the overall strain is given by

(13)
$$\frac{\ell - \ell_i}{\ell_i} = \frac{N - N_0}{A_s E} ,$$

where $N - N_0$ is the increase in tension due to the longitudinal strain induced by the sag. The length of the deformed section can be found from the approximate expression:

(14)
$$\ell = \ell_0 + \int_0^{\ell_0/2} \theta^2 d\alpha,$$

which is quite valid as long as $|\theta| \ll 1$.

Longitudinal Slippage at the Ends of the Pipe.

The quantity ℓ_i is not necessarily the same as ℓ_0 , because if longitudinal slippage of the pipe occurs at the ends, then the original length of the section between points (1') and (1) will be greater than ℓ_0 and the resulting tension in the pipe will be lower than that for the case of no slippage. The amount of slippage will depend upon the longitudinal restraint offered by the stronger sediment adjacent to the zone in which sag occurs. If the sediment exerting this restraining force is perfectly rigid then no slippage will occur and ℓ_i will equal ℓ_0 . If, on the other hand, the sediment at the ends of the pipe offers very little restraint, then ℓ_i will be nearly the same as ℓ and the resulting value of $(N - N_0)$ will be small.

The amount of slippage at each end of the sagging section is $(\ell_i - \ell_0)/2$. This slippage is proportional to the increase in tension at the ends of the sagging section. The slippage is also proportional to the effective length of pipe, adjacent to the sagging section, which undergoes elongation. This effective length is determined the final balance existing between the longitudinal restraint exerted by the surrounding sediments, and the increase in tension ($N - N_0$) at the ends of the sagging section. From these considerations it can be shown that the following approximation is applicable

(15)
$$\ell_i - \ell_0 = \frac{(N - N_0)^2}{f_r A_s E} ,$$

where f_r represents the maximum longitudinal restraining force per unit length of pipe offered by the sediments adjacent to the weak zone.

If the pipe is buried in the sediments in the adjacent sections, then f_r can be expressed in terms of the ultimate shear strength, τ_u', of the relatively strong sediment as follows:

$$(16) \qquad f_r = \pi D \tau_u',$$

where it is assumed that shear occurs at or near the surface of the pipe of overall diameter D.

For a pipe lying on the bottom f_r is equal to the coefficient of friction between the pipe and bottom multiplied by the submerged weight of pipe per unit length.

The Characteristic Dimensionless Parameters

In order to make the functional relationships existing between the basic variables of the problem as simple as possible it is convenient to introduce the dimensionless parameters given in the Table I.

Table I

Eq.	Name	Symbol		Definition
(17)	Bending moment factor	m	$=$	$\dfrac{M}{w \ell_o^2}$
(18)	End shear factor	p	$=$	$\dfrac{V_1}{w \ell_o}$
(19)	Tension factor	n^2	$=$	$\dfrac{N \ell_o^2}{EI}$
(20)	Flexibility parameter	q	$=$	$\dfrac{w \ell_o^4}{EI r}$ *

The definition (17) can be applied to the moments at the middle and at the ends. The quantity

$$(21) \qquad m_o = \frac{M_o}{w \ell_o^2}$$

---

* The quantity r is the radius of gyration of the cross section as defined previously and equals $\sqrt{I/A_s}$.

is the bending moment factor at the point of maximum sag, and

(22)
$$m_1 = \frac{M_1}{w \, l_o^2}$$

is the bending moment factor at the ends of the sagging section.

Likewise, as a special case of (19):

(23)
$$n_o^2 = \frac{N_o \, l_o^2}{EI} \, ,$$

which is the intial tension factor.

By making use of equations (7) to (11), the following relations can be established:

(24)
$$\phi = \frac{1}{2} - \frac{n^2}{q} \cdot \frac{l_o}{r} \, \theta_1 \, ,$$

(25)
$$m_o = \frac{-\phi}{n \sinh n/2} + \frac{1}{n^2} \, ,$$

(26)
$$m_1 = \frac{-\phi}{n \tanh n/2} + \frac{1}{n^2} \, ,$$

and

(27)
$$\frac{y_m}{r} = \frac{q}{n^2} \left[\frac{1}{8} + \phi \, \frac{1 - \cosh n/2}{n \sinh n/2} \right] \, ,$$

where y_m is the maximum vertical deflection or simply the sag. Special forms of these expressions are given below for the two commonly visualized end conditions.

Case I: Rigid Ends with Zero Slope

This condition is illustrated schematically in Figure 1D and represents the situation for which $\theta_1 = 0$. In this case the equations (24) to (27) take the form:

(24a)
$$\phi = \frac{1}{2} \, ,$$

(25a)
$$m_0 = -\frac{1}{2n \sinh n/2} + \frac{1}{n^2},$$

(26a)
$$m_1 = -\frac{1}{2n \tanh n/2} + \frac{1}{n^2},$$

and

(27a)
$$\frac{y_m}{r} = \frac{q}{n^2}\left[\frac{1}{8} + \frac{1 - \cosh n/2}{2n \sinh n/2}\right].$$

The maximum bending moment factor (and hence the maximum bending moment)
occurs at the ends for this condition and is given by m_1 .

Case II: Ends Free to Turn

In this case $m_1 = 0$, which implies a maximum slope (or inflection)
at the ends of the sagging section. For this condition:

·(24b)
$$\frac{l_0}{r} \theta_1 = \frac{q}{n^2}\left(\frac{1}{2} - \frac{\tanh n/2}{n}\right) *,$$

(25b)
$$m_0 = \frac{1}{n^2}\left(1 - \frac{1}{\cosh n/2}\right),$$

(26b)
$$p = \frac{1}{n} \tanh n/2,$$

and

(27b)
$$\frac{y_m}{r} = \frac{q}{n^2}\left[\frac{1}{8} + \frac{1 - \cosh n/2}{n^2 \cosh n/2}\right].$$

In this case the maximum bending moment occurs at the center of sag.

The Relation between the Tension Factor and Flexibility

It should be noted from the relations above that the bending
moment factors m_0 and m_1 and also the shear factor p are fully

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* Note that l_0/r represents the slenderness ratio parameter, which is
the critical variable in the stability theory of columns.

determined by the tension factor n^* . This factor must be determined from the flexibility parameter q , the end conditions, and the initial tension. Equations (10), (13), (14), (15) together with (25a) or (25b) yield the additional relations required:

(28)
$$q = k_1 k_2 f(n),$$

where

(29)
$$k_1 = \sqrt{1 - n_0^2/n^2} ,$$

(30)
$$k_2 = \sqrt{1 + \Delta q^{-3/4}(n^2 - n_0^2)} ,$$

and

(31)
$$\Delta = \frac{w}{f_r} \left(\frac{EA_s}{wr}\right)^{1/4} = \frac{w}{f_r} \left(\frac{EI}{wr^3}\right)^{1/4} .$$

The function $f(n)$ depends upon the end condition.* For case I ($\theta_1 = 0$):

(32) $$f(n) = f_1(n) = \sqrt{24} \, n^4 \left\{ n^2 - 9n + 24 + 3n \left[\frac{n + 3(1 - e^{-n})}{1 - \cosh n}\right]\right\}^{-1/2} ,$$

while for case II ($m_1 = 0$):

(33) $$f(n) = f_2(n) = \sqrt{24} \, n^4 \left\{ n^2 - 24 + \frac{12}{n}\left(\frac{n - 5 \sinh n}{1 + \cosh n}\right)\right\}^{-1/2} .$$

The quantity Δ is a dimensionless parameter which may be referred to as the slippage coefficient. The slippage is large when the restraining force f_r is small compared with w . If the ends of the pipe are held so that no slippage occurs, then $k_2 = 1$.

If both Δ and n_0 are zero then $k_1 = k_2 = 1$ and (28) reduces to

(28a)
$$q = f_1(n) \quad \text{for} \quad \theta_1 = 0 ,$$

or

(28b)
$$q = f_2(n) \quad \text{for} \quad m_1 = 0 .$$

---

*   . See Table VIII in the Appendix for tabulated values of $f(n)$ computed for different values of n ranging from .01 to 1,000.

The function $f_1(n)$ [i.e., q for $n_o^2 = 0$, $\Delta = 0$, $\theta_1 = 0$] is shown graphically in Figure 3 by the dashed curve. The function $f_2(n)$ is shown graphically by the full curve labeled $n_o^2 = 0$ in the same graph. By means of this graph it is readily possible to determine n^2 corresponding to a given value of q for the case of no initial tension and no slippage. Curves of n^2 versus q corresponding to four different finite values of n_o^2 are also shown in Figure 3 for the end condition $m_1 = 0$.

The expressions for $f(n)$ in equations (32) and (33) reduce to simple forms for the limiting conditions of very small and very large values of n :

(35a,b)
$$\lim_{n \to 0} \frac{f_1(n)}{n} = \sqrt{\frac{9!}{6}} \doteq 246 \; ; \qquad \lim_{n \to 0} \frac{f_2(n)}{n} = \sqrt{\frac{8!}{17}} \doteq 48.7 \; ;$$

and

(36)
$$\lim_{n \to \infty} \frac{f_1(n)}{n^3} = \lim_{n \to \infty} \frac{f_2(n)}{n^3} = \sqrt{24} \doteq 4.90 \; .$$

Thus for the case of no end slippage and no initial tension

(37)
$$n^2 = \frac{q^{2/3}}{(24)^{1/3}} \; ,$$

for large values of q . Equation (37) is a good approximation if:

$$q > 10^6 \quad , \qquad \text{for} \quad \theta_1 = 0 \; ;$$

or

$$q > 10^4 \quad , \qquad \text{for} \quad m_1 = 0 \; .$$

Physically this means that for sufficiently large values of q , the pipe acts essentially as a flexible cable. Equation (37) can be put in the form

(37a)
$$N = \left[\frac{E A_s}{24} (w l_0)^2 \right]^{1/3} \; ,$$

which is the expression for the tension in a flexible cable of length l_0 , provided that the sag is very small relative to l_0 .

If slippage or initial tension is appreciable, or if q is not sufficiently large, then the general expressions for $f(n)$ must be used and the factors k_1 and k_2 must be taken into consideration. A more complete graph could be constructed so that n^2 could be readily determined from q, n_o^2 and Δ. Lacking such a complete graph, the evaluation of n^2 can be carried out by successive approximation without too much difficulty (see examples illustrating this procedure at the end of part I).

It will be noted from Figure 3 that the scales for n^2 and q^2 are logarithmic, allowing for a wide range of values in both parameters. The general curves approach the limiting asymptotic relations for very large or very small values of q. The limiting asymptotes appear as straight lines on the graph, thus making it a simple matter to extrapolate the curves for values lying outside the range of the graph.

Determination of the Bending Moment Factor

For the end condition of case I m_1 is of prime concern, while for case II m_o is the important factor for determination of the maximum bending moment in the pipe. As indicated above, these factors are functions of n^2 only and are represented graphically in Figure 4. The factor m_1 corresponding to $\theta_1 = o$ is represented by the dashed curve, while m_o for the case of $m_1 = o$ is represented by the full curve.

The limiting expressions for these functions can be obtained from equations (26a) and (25b); see Table II below:

Table II
Limiting values of m_o and m_1

Case I ($\theta_1 = o$)		Case II ($m_1 = o$)	
Value of m_1	Condition	Value of m_o	Condition
$-\dfrac{1}{12}$	$n^2 < 3$	$\dfrac{1}{8}$	$n^2 < 5$
$-\dfrac{1}{2n}$	$n^2 > 1000$	$\dfrac{1}{n^2}$	$n^2 > 500$
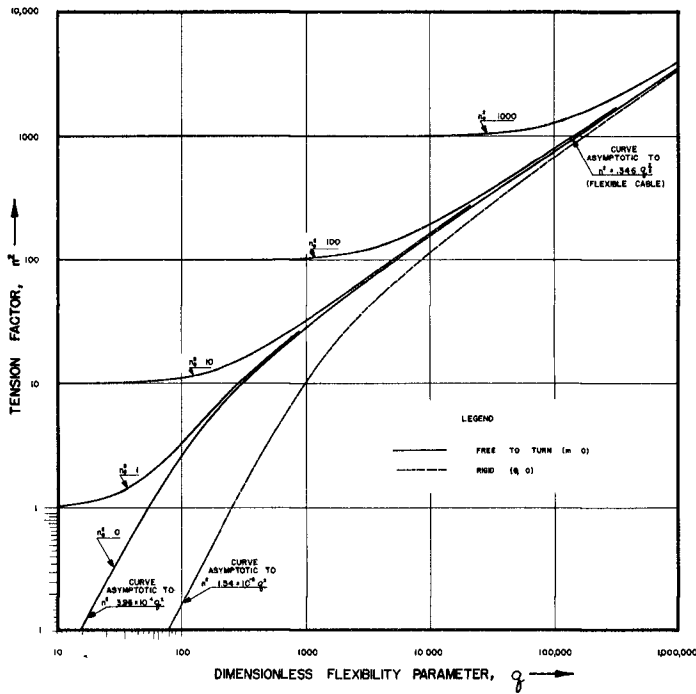
Fig. 3. Dimensionless tension factor n^2, as a function of the dimensionless flexibility parameter, for different values of initial tension as indicated, and for two different end conditions. All curves apply to the case of no end slippage ($\Delta = 0$).
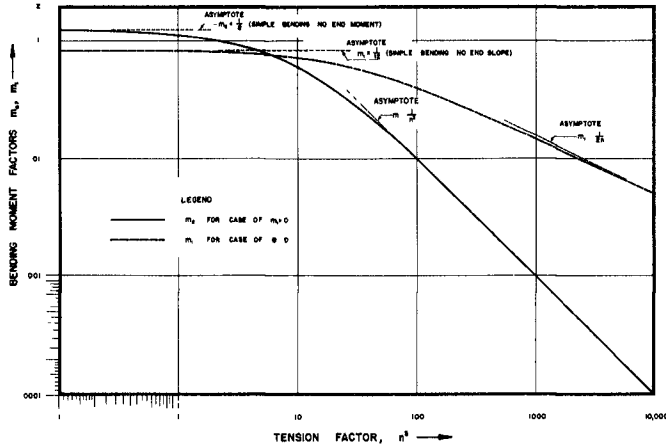
Fig. 4. Maximum bending moment factors, m_0 and m_1 (m_0, for the case of no end moment, and m_1 for the case of zero end slope) as functions of the tension factor, n^2.

The limiting expressions for M_1 and M_o in the case of small tension are consequently:

$$(38) \qquad M_1 = -\frac{1}{12} w \ell_o^2 \quad , \quad for \quad \theta_1 = 0 ,$$

and

$$(39) \qquad M_o = \frac{1}{8} w \ell_o^2 \quad , \quad for \quad M_1 = 0 ,$$

which are the same expressions obtained from the theory of simple bending. For very large tension, on the other hand, the bending moments are a very small fraction of $w \ell_o^2$. In case II, the tension becomes the prime factor in the determination of the stress in the pipe, when n^2 is large.

Figure 5 shows the bending moment factors as a function of the flexibility for the case of no end slippage. The dashed curve represents m_1 versus q for $\theta_1 = 0$ and $n_o^2 = 0$. The full curves represent m_o versus q for the different values of n_o^2 indicated, for the condition $m_1 = 0$.

Practical Forms of the Equations for Slope and Relative Sag

The equations (27a), (24b) and (27b) can be simplified by use of (28). The formula for the slope in case II becomes

$$(40) \qquad \frac{\ell_o}{r} \theta_1 = k_1 k_2 k_3 n ,$$

where k_1 and k_2 are the same as defined in (29) and (30) and k_3 is a coefficient which depends upon n . It can be shown that for the entire range of n :

$$(small\ n) \sqrt{\frac{70}{17}} < k_3 < \sqrt{6} \quad (large\ n).$$

The average value of k_3 is about 2.24, and the following approximation will yield values of θ_1 which are never more than 10 per cent in error:

$$(40a) \qquad \theta_1 \doteq 2.24 \, k_1 k_2 \frac{r}{\ell_o} n , \quad (m_1 = 0).$$

The formula for the relative sag can be simplified in a similar manner:

$$(41) \qquad \frac{y_m}{r} = k_1 k_2 k_4 n .$$

The coefficient k_4 is determined by n , however for the entire range of n;

$$(\text{large } n) \qquad \sqrt{3/8} \; < \; k_4 \; < \; \sqrt{105/256} \qquad (\text{small } n).$$

The average value of k_4 is about 0.626 for either of the end conditions examined here. This means that the following approximation will yield values of y_m which are accurate to within about 2 per cent:

$$(41a) \qquad y_m \doteq 0.626 \, k_1 k_2 \, r \, n , \qquad \begin{Bmatrix} \theta_1 = 0 \\ \text{or} \\ m_1 = 0 \end{Bmatrix} .$$

It will be noted that

$$k_1 n = \sqrt{n^2 - n_o^2} ,$$

so that both θ_1 and y_m are proportional to the square root of the increase in tension in the pipe due to sag. Furthermore, if there is no end slippage the coefficient k_2 is unity.

For small values of q the equations (27a) and (27b) reduce to the forms:

$$(42a) \qquad y_m = \frac{1}{384} \frac{w l_o^4}{EI} , \qquad \text{for } \theta_1 = 0 ;$$

and

$$(42b) \qquad y_m = \frac{5}{384} \frac{w l_o^4}{EI} , \qquad \text{for } m_1 = 0 ,$$

provided that there is no initial tension. These equations are also obtained from the theory of simple bending.

Limit of Application of the Simple Theory of Bending

The simple theory of bending of a beam under the action of a uniform load per unit length is seen to be a limiting case of the theory
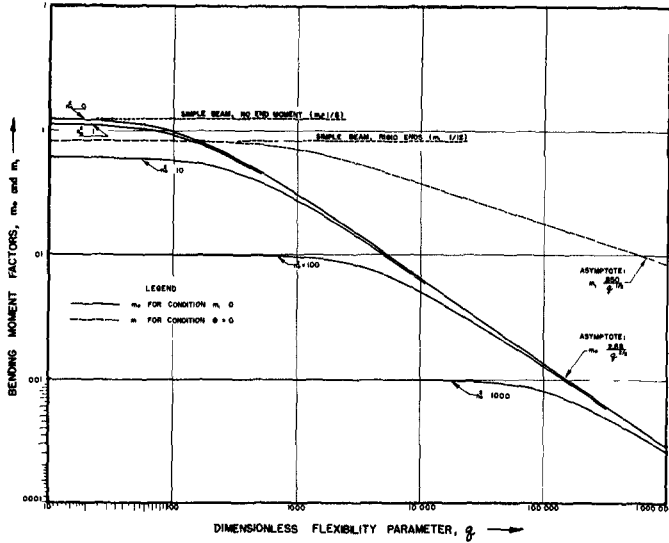
Fig. 5. Maximum bending moment factors (m_0 and m_1) as functions of the flexibility parameter q, for different values of initial tension factor, n_0^2, as indicated. All curves constructed for the case of no end slippage ($\delta = 0$).
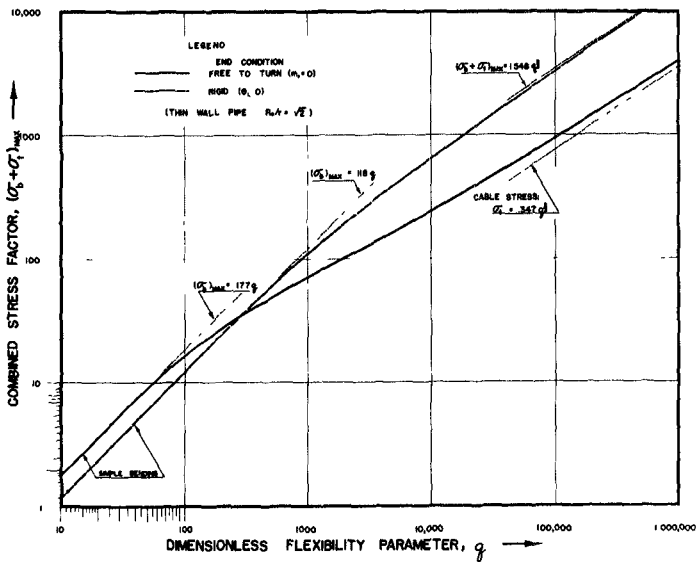
Fig. 6. Maximum combined stress factors, for the two indicated end conditions, as functions of the flexibility parameter q. Both curves apply to the case of no end slippage and no initial tension for a pipe with relatively thin walls compared to its diameter. Limiting relations for very small q and very large q indicated by light dashed lines.

of combined flexure and elongation given here. The question of whether or not the generalized theory is required for a solution to the problem depends upon the value of the dimensionless flexibility parameter and the initial tension factor.

It is evident from Figure 5 that when an initial tension corresponding to a value of n_o^2 greater than unity exists, then the simple theory of bending is not valid. Furthermore, Figure 5 indicates that, for $n_o^2 = 0$, there is also a practical limit of flexibility beyond which the simple theory of bending becomes invalid. For a 10 per cent tolerance of error in the simple theory, the upper limit of flexibility (q_c) in the application of the simple theory is:

(43a)
$$q_c = 700, \text{ for } \theta_1 = 0 \quad \text{(rigid ends)},$$

or

(43b)
$$q_c = 50, \text{ for } m_1 = 0 \quad \text{(ends free to turn)}.$$

From equations (42a) and (42b) and the critical limits of q given above, it is evident that the upper limits of the relative deflection for which the simple theory is valid are:

(44a)
$$\left(\frac{y_m}{r}\right)_c \doteq 1.8, \text{ for } \theta_1 = 0,$$

and

(44b)
$$\left(\frac{y_m}{r}\right)_c \doteq 0.65 \text{ for } m_1 = 0,$$

That is, if the simple theory is to apply, then the maximum deflection must be less than the order of magnitude of the radius of gyration of the cross section.

It is possible, for given pipe specifications, to interpret the above criteria, for the allowable use of simple bending theory, in terms of the length (l_o)$_c$ for different values of net loading w . Table III gives this information for two different pipe sizes, and for values of net load from 1 lb. per ft. to an extreme value of 1000 lbs. per ft. The load is governed by the weight of pipe (including ballast, if any), the weight of fluid in the pipe, and the characteristics of the sediment as discussed above, and is not necessarily governed by the pipe specifications alone. The stress S_b induced by simple bending under the conditions stated is also included in Table III to indicate that, in most of the situations

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represented, this is not the governing factor so far as the validity of the simple theory is concerned. In other words, taking the 20.5 inch pipe with $w = 10$ lbs. per ft. as an example, it is apparent that there is considerable latitude for increase in $\mathcal{L}_o$ beyond the critical value of 210 feet given for case II, so far as the stress is concerned. However, the generalized theory must be used in order to determine the sag characteristics for $\mathcal{L}_o$ beyond this value. It can be shown, in fact, that for sufficient length this pipe would fail essentially in elongation rather than in bending, under conditions of free sag with the load of 10 lbs. per ft.

Table III

Critical values of pipe length #, corresponding to different
net loads per unit length, beyond which simple bending theory
is invalid; and bending stress corresponding to these conditions

(a) 10 inch O.D. steel pipe: (1/2 inch walls) $I = 169$ in.^{4} $A_s = 14.9$ in.^{2} $r = 3.36$ in.				
	Case I: $\theta_1 = 0$		Case II: $m_1 = 0$	
w	$(\mathcal{L}_o)_c$	S_b	$(\mathcal{L}_o)_c$	S_b
lb./ft.	ft.	psi	ft.	psi
1	265	2,080	149	985
10	149	6,580	84	3,130
100	84	20,900	47	9,800
1000	47	(60,400)*	26	30,000

(b) 20.5 inch O.D. steel pipe: (3/4 inch walls) $I = 2270$ in.^{4} $A_s = 46.5$ in.^{2} $r = 6.98$ in.				
	Case I: $\theta_1 = 0$		Case II: $m_1 = 0$	
w	$(\mathcal{L}_o)_c$	S_b	$(\mathcal{L}_o)_c$	S_b
lb./ft.	ft.	psi	ft.	psi
1	664	2,000	372	938
10	372	6,250	210	2,980
100	210	20,000	118	9,440
1000	118	(63,100)*	66	29,500

\#   Based upon the assumption that there is no initial tension;
E taken as 30×10^6 psi.

*   Values in parenthesis represent stresses beyond the endurance
limit for ordinary steels.

Induced Stresses

The bending moment induces a non-uniform normal stress across the section of the pipe. The maximum value of this stress, S_b , occurs in the material farthest from the neutral surface of bending, and is given by

$$(45) \qquad S_b = \frac{|M| R_o}{I} = \frac{|m| w l_o^2 R_o}{I} ,$$

where R_o is the outside radius of the steel pipe. The bending stress is zero at the neutral surface of bending and varies from $- S_b$ at the concave side to $+ S_b$ at the convex side of bending. The tension induced by axial elongation of the pipe gives rise to a uniform stress, S_t , given by

$$(46) \qquad S_t = \frac{N}{A_s} = \frac{n^2 E I}{A_s l_o^2} .$$

Thermal stress associated with restraint of axial elongation or contraction of the pipe is included in this term, since in determining N thermal effects must be taken into account in the term N_o .

The vertical shearing force gives rise to a non-uniform shear stress at each section of the pipe. The mean value of this shear stress is

$$(47) \qquad \overline{S}_s = \frac{V}{A_s} = \frac{p w l_o}{A_s} .$$

The transverse shear stress varies from zero in the steel farthest from the neutral surface to a maximum at the neutral surface. For a pipe of standard wall thickness relative to the diameter, the maximum shear stress is approximately $2 \overline{S}_s$.

The fluid pressure within the pipe will give rise to still another stress due to the circumferential elongation of the pipe. This stress is called the hoop stress, S_h , and is a uniform normal stress which is perpendicular to the normal stresses induced by axial elongation and bending. If Δp represents the difference in pressure between the inside and outside of the pipe, then

$$(48) \qquad S_h = \frac{D_i}{D_o - D_i} \Delta p ,$$

where D_o and D_i are the outside and inside diameters of the pipe, respectively.

The formula, for the most severe <u>combined stress</u> at a given section of the pipe depends upon the value of the criterion parameter $4\,\overline{S}_s^{\,2}/S_b$. Table IV gives the appropriate expression for the governing stress (either $S_{t,m}$ or $S_{s,m}$) for three different conditions imposed upon the criterion parameter.

<u>TABLE IV</u>

The Governing Combined Stress
and the Criterion for its Choice

Condition	Governing Combined Stress
(A) $\quad 0 \leq \dfrac{4\,\overline{S}_s^{\,2}}{S_b} < S_1$	$S_{t,m} = S_b + S_t$
(B) $\quad S_2 > \dfrac{4\,\overline{S}_s^{\,2}}{S_b} > S_1$	$S_{t,m} = \dfrac{1}{2}\left\{(S_t+S_h) + \sqrt{(S_t-S_h)^2 + \text{...}}\right.$
(C) $\quad S_2 < \dfrac{4\,\overline{S}_s^{\,2}}{S_b} > S_1$	$S_{s,m} = \dfrac{1}{2}\sqrt{(S_t-S_h)^2 + 16\,\overline{S}_s^{\,2}}$
where $\quad S_1 = (S_b+S_t) - S_h \quad ; \quad S_2 = \dfrac{S_t\,S_h}{S_b}$	

It will be noted from Table IV that condition (C) implies that the maximum shear stress $S_{s,m}$ governs. This presumes that the yield limit of stress in shear is just half that in tension for the pipe steel.

Under conditions (A) the pipe would fail in tension at a point farthest from the neutral surface, provided that $S_{t,m}$ were great enough. Under condition (B) the pipe would fail in tension at the neutral surface, along a plane which forms an angle of less than 90° with the neutral surface. Under condition (C) failure, if it occurred, would manifest itself by shear at the neutral surface, along a plane which forms an angle of less than 90° with the neutral surface.

The position along the pipe line at which the maximum stress occurs depends upon the end conditions. The stress S_b is a maximum at the ends of the sagging section for the case of zero end slope; while in the case of ends free to turn, the maximum value of S_b would occur at the center of sag. The stress $\overline{S_s}$, on the other hand, is a maximum at the ends in both cases, but its magnitude depends upon the end condition. The stresses S_t and S_h are presumed to be independent of position along the pipe.

For practical purposes, the condition (A) can be presumed for nearly all cases of pipe sag, and the governing stress therefore is

$$(49) \qquad S_{t,m} = S_b + S_t ,$$

where S_b is the value occurring at the position of maximum flexure. The validity of this assumption, however, can be checked by computing $4\left(\frac{s}{s}\right)^2 / S_b$. This must be less than the value of $(S_b + S_t) - S_h$, otherwise $S_b + S_t$ is not the maximum combined stress.

The Dimensionless Stress Parameters

It is convenient to introduce the following dimensionless stress parameters:

$$(50) \qquad \sigma_b = \left(\frac{l_e}{r}\right)^2 \frac{S_b}{E} ,$$

and

$$(51) \qquad \sigma_t = \left(\frac{l_e}{r}\right)^2 \frac{S_t}{E} .$$

From equations (45) and (46) it can be shown that the stress parameters are related to the characteristic parameters m, n, and q as follows :

$$(52) \qquad \sigma_b = \frac{R_e}{r} m q ,$$

and

$$(53) \qquad \sigma_t = n^2 .$$

The maximum combined stress for condition (A) is therefore

$$(54) \quad S_b + S_t = \left(\frac{r}{l_o}\right)^2 E \left(\sigma_b + \sigma_t\right) = \left(\frac{r}{l_o}\right)^2 E \left(\frac{R_o}{r} mq + n^2\right).$$

The quantity $(\sigma_b + \sigma_t)$ can be shown graphically as a function of q for given values of n_o^2, Δ, and R_o/r.

For all practical purposes, the value of R_o/r for most pipes may be taken as $\sqrt{2}$. This is theoretically correct for a circular pipe with thin walls. However, if greater accuracy is desired, the following relation can be used:

$$(55) \quad \frac{R_o}{r} = \frac{2}{\sqrt{1 + \left(R_i / R_o\right)^2}},$$

where R_i is the inside radius.

Figure 6 has been constructed using $R_o/r = \sqrt{2}$, for the case of $n_o^2 = 0$ and $\Delta = 0$. The value of the combined stress factor $(\sigma_b + \sigma_t)$ given as a function q corresponds to the combined stress at the point of maximum bending in the sagging pipe. Curves for the two investigated end conditions, $\theta_1 = 0$ and $m_1 = 0$, are shown. In the case of ends free to turn, the stress factor approaches that for simple bending ($\sigma_t = 0$) for very small values of q. For very large values of q, on the other hand, the combined stress factor approaches that corresponding to the stress in a flexible cable.

In the case of $\theta_1 = 0$, the curve approaches that for simple bending, for low values of q. However, at high values of q the values of $(\sigma_b + \sigma_t)$ are considerably greater than the corresponding values for the case of $m_1 = 0$.

In the general problem for which $n_o^2 \neq 0$ and $\Delta \neq 0$, it is necessary to make use of the relations (52) and (53) in order to compute the combined stress factor. The value of m to be used in relation (52) is the maximum factor for the particular end condition.

Table IV gives the limiting expressions for σ_b, σ_t, and $(\sigma_b + \sigma_t)$ for the case of no initial tension and no end slippage.

Table V

Limiting expressions for the bending and the tensile stress factors
for the case of $n_o^2 = 0$ and $\Delta = 0$

End Condition	Stress factors		Stress factors and combined stress factor for a pipe with thin walls $R_o/t = \sqrt{2}$			Values of q for which limiting relation applies
	σ_b	σ_t	σ_b	σ_t	$\sigma_b + \sigma_t$	
Case I $(\theta_1 = 0)$	$\dfrac{1}{12}\dfrac{R_o}{t}q$	$\dfrac{q^2}{60{,}480}$	$.118q$	$1.65 \times 10^{-5} \times q^2$	$.118q$	$q < 1000$
	$\left(\dfrac{3}{8}\right)^{1/6}\dfrac{R_o}{t}q^{2/3}$	$\dfrac{q^{2/3}}{(24)^{1/3}}$	$1.201q^{2/3}$	$.347q^{2/3}$	$1.548q^{2/3}$	$q > 10^6$
Case II $(m_1 = 0)$	$\dfrac{1}{8}\dfrac{R_o}{t}q$	$\dfrac{17q^2}{40{,}320}$	$.177q$	$4.21 \times 10^{-4} \times q^2$	$.177q$	$q < 100$
	$\dfrac{(24)^{1/3}R_o}{t}q^{1/3}$	$\dfrac{q^{2/3}}{(24)^{1/3}}$	$4.08q^{1/3}$	$.347q^{2/3}$	$.347q^{2/3}$	$q > 10^7$

Resume of Theory of Free Sag

The independent dimensionless variables in the problem of pipe sag
are the flexibility parameter q, the initial tension factor n_o^2,
and the end slippage coefficient Δ. Given these, the factor n^2 can
be ascertained, and consequently m_o, m_1, p, θ, l_o/t, y_m/t, σ_t,
σ_b, and $\sigma_t + \sigma_b$ can be evaluated for a given end condition. The basic
quantities sought can then be found from the dimensionless parameters by
applying the definitions of these parameters given by relations (17) to
(23) together with (50) and (51). The basic physical quantities which
must be known in order to determine the actual tension, bending moments,
etc., are summarized in Table VI.

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Table VI

Category	Physical Quantity	Units in the ft.-lb.-sec. system
loading factors	$\begin{cases} N_o \\ w \\ f_r \end{cases}$	lbs. lbs. per ft. lbs. per ft.
span of weak zone	$\begin{cases} \ell_o \end{cases}$	ft.
pipe specification factors	$\begin{cases} E \\ I \\ A_s \end{cases}$	lbs. per sq. ft. ft.^{4} ft.^{2}

The problem is thus formally solved, but only for the two specific end conditions chosen, and subject to the restrictions implied in the basic assumptions (principally numbers 1, 2, 9 and 10).

The theory of pipe sag presented above is not necessarily restricted to the case of sag into weak sediments, but may apply to a wide class of situations encountered in the installation of the pipe line. The situations encountered in nature may be classified as either simple or complex from the standpoint of the application of theory. The problem falls in the simple category if the conditions are such that the assumptions in the theory are fulfilled for all practical purposes. If this is not the case, then the problem is complex. However, it may be possible, by proper separation of the problem into various parts, to apply the theory in modified form. Such a technique must be used in analyzing the situation of restricted sag, where a portion of the sagging section of pipe is supported by firm sediments after a certain amount of sinking occurs.

Restricted Sag

The theory of free sag is subject to the condition

$$y_m < h,$$

where h is the vertical distance from the ends of the sagging pipe section to the bottom of the weak sediment zone. Under certain conditions the sag may be great enough that the central portion of the sagging section rests upon the firmer sediments at the base of the weak zone (see Figure 2A). In this case the portion of pipe subject to free sag is not ℓ_o but is a smaller length ℓ_o'. The equivalent free sag problem for

the length ℓ_o' is represented graphically in Figure 2B. It is considered that the slope of this section is small enough that the unit load, w, is nearly normal to the pipe as previously assumed. The vertical scale in the schematic diagrams of Figure 2 is greatly exaggerated.

For the case of zero moment at the ends of section ℓ_o', the equivalent end slope, referred to the rotated coordinates in Figure 2B, is

$$(56) \qquad \theta_1' = \frac{h}{\ell_o'} .$$

The section ℓ_o' is subject to an effective initial tension factor $(n_o')^2$ given by

$$(57) \qquad (n_o')^2 = n_o^2 \left(\frac{\ell_o'}{\ell_o}\right)^2 + \frac{\ell_o'}{\ell_o}\left(\frac{h}{r}\right)^2 ,$$

where n_o^2 is the initial tension factor of the straight pipe.

By making use of equations (28), (29), and (40a); together with (56) and (57), for the special case of $n_o^2 = 0$ and no end slippage, the following important relations are obtained:

$$(58) \qquad q \doteq 279 \frac{h}{r} \frac{f_2(n')}{n'} \left[\frac{5(n')^2}{(h/r)^2} - 1\right]^{-4} ,$$

and

$$(59) \qquad \frac{\ell_o'}{\ell_o} \doteq \frac{(n')^2}{(h/r)^2} - \frac{1}{5} ,$$

where

$$(60) \qquad (n')^2 = \frac{N(\ell_o')^2}{EI} .$$

The function $f_2(n')$ is that given by equation (33), where n is replaced by n'. The graph of this function, as already noted, is represented by the full curve labled $n_o^2 = 0$ in Figure 3; in this graph, enter with $(n')^2$ on the vertical scale and read $f(n')$ on the horizontal scale.

The quantities $(n')^2$ and ℓ_o are the key factors to determine, because once these are determined, the tension and bending moment can easily be evaluated. Equation (58) is not explicit in n' and therefore must be solved for n' by successive approximation and/or graphically. The maximum possible range of ℓ_o'/ℓ_o is 0 to 1/2, and therefore must lie in the range:

$$(61) \qquad 0.20 \left(\frac{h}{r}\right)^2 < (n')^2 < 0.70 \left(\frac{h}{r}\right)^2 ,$$

according to equation (59). This serves as a guide in the selection of values for a solution to equation (58). The upper limit $0.70 (h/r)^2$ corresponds to $\ell_o' = \ell_o/2$. This represents the condition for tangency of the pipe at the bottom, with <u>zero moment</u> at the point of contact. In the case where the pipe is tangent to the bottom but receives no support, $(n')^2$ is equal to $0.64(h/r)^2$ according to equation (41a); under this condition a bending moment does exist at the point of contact.

The maximum bending moment factor for the condition of no end moment is m_o'. This can be determined from equation (256) by replacing n by n' or can be obtained from the solid curve given in Figure 4 by entering with $(n')^2$ on the horizontal scale and reading m_o' on the vertical scale. The bending moment can then be computed from the relation

$$(62a) \qquad M_o' = m_o' \, w (\ell_o')^2.$$

For the case of rigid ends of the section ℓ_o, the problem becomes complex, for in this event the section ℓ_o' is no longer symmetrical with reference to the rotated coordinates of Figure 2B. One end is rigid and subject to a maximum bending moment while the other end is free of moment. However, as a first approximation the equations (24) and (26) can be used to determine the maximum bending moment factor. The angle θ_1' for this case will be $-h/\ell_o'$; therefore

$$(63) \qquad m_1' \doteq - \frac{\left[\frac{1}{2} + \frac{(n')^2}{q'} \frac{h}{r}\right]}{n' \tanh \frac{n'}{2}} + \frac{1}{(n')^2} ,$$

where

$$(64) \qquad q' = q \left(\frac{\ell_o'}{\ell_o}\right)^4.$$

The values of n' and l_o' , obtained from equations (58) and (59) can be used in (63) and (64) as a first approximation. Finally the end moment can be found from

$$(62\text{b}) \qquad M_1' = m_1' \, w \, (l_o')^2 ;$$

and the tension can be found from equation (60).

DEFORMATION OF A PIPE LINE IN AN ELASTICALLY DEFORMED MATERIAL

In general the relatively strong material which supports the pipe at the ends of the sagging section will be deformed itself. This may be a quasi-elastic deformation if the material is sufficiently strong, or it may be predominantly a plastic deformation. The condition of rigid ends is a hypothetical situation representing the limiting case of elastic deformation of a material of infinite strength and infinite elastic modulus. On the other hand, the condition of zero moment at the ends of the section l_o is a special case of plastic deformation of the supporting material, where the net upward unit load on the pipe is the same in magnitude as the downward unit load in the weak sediment zone.

The situation of elastic deformation of a relatively strong supporting material is considered here. By examining the mutual distortion of pipe and supporting material under the action of a known total load on the pipe, it is possible to arrive at a "modified rigid end" condition, which makes the determination of stresses associated with pipe sag more realistic.

The Basic Equations

The equation for balance of vertical forces on a small section of pipe is given by

$$(65) \qquad EI \frac{d^4 y'}{dx'^4} - N \frac{d^2 y'}{dx'^2} = f ,$$

where f is the net force per unit length acting in the direction of y' . As before y' is taken vertically upwards. Equation (65) is subject to the assumptions (4) to (10) inclusive given in the preceding section, but the restrictions imposed by (1), (2), and (3) are dropped.

The assumption which is made in place of (1) and (2) is that

$$(66) \qquad f = - E_e \, y' \, , \quad \text{or} \quad E_e = \left| \frac{f/D}{y'/D} \right| > 0 \, ,$$

where E_e is the effective elastic modulus of the sediment, and y' is measured upward from the equilibrium position of the pipe.

The other principal assumption is that the supporting material, which is subject to elastic deformation, is of semi-infinite extent in the positive x' direction. The origin is shifted horizontally to the end of the sagging pipe section, so that $x' = 0$ is the dividing line between the weak sediment and the relatively stronger, supporting material adjacent to it. Furthermore, it is presumed that y' approaches zero for very large values of x', i.e. it reaches the equilibrium position at great distance from the end at which the sag load is applied.

Under these conditions, the solution of (65) is given by one of the following equations:

$$(67a) \qquad y' = \frac{y_i'}{\sin \nu c} \, e^{-\beta x'} \sin \nu (c - x') \, , \quad \text{for } \gamma < 1 \, ;$$

$$(67b) \qquad y' = \frac{y_i'}{c} \, e^{-\alpha x'} (c - x') \, , \quad \text{for } \gamma = 1 \, ;$$

$$(67c) \qquad y' = \frac{y_i'}{e^{n_1 c} - e^{n_2 c}} \left[e^{n_1 (c - x')} - e^{n_2 (c - x')} \right] , \quad \text{for } \gamma > 1 \, ;$$

where

$$(68) \qquad \gamma = \frac{N}{2 \sqrt{EI \, E_e}} \, ,$$

$$(69) \qquad \alpha = \left(\frac{E_e}{EI} \right)^{1/4} \, ,$$

$$(70a,b,c) \qquad \beta = \alpha \sqrt{\frac{1 + \gamma}{2}} \, , \quad \nu = \alpha \sqrt{\frac{1 - \gamma}{2}} \, , \quad \nu' = \alpha \sqrt{\frac{\gamma - 1}{2}} \, ,$$

(71a,b)
$$n_1 = \beta + \nu', \quad n_2 = \beta - \nu'.$$

The constant y'_1 represents the vertical displacement of the pipe at the end $x' = 0$, and c represents the position of $y' = 0$. The quantity γ is a dimensionless quantity which will be referred to as the tension parameter. The quantities α, β, ν, ν', n_1, and n_2 have the dimensions of wave number (reciprocal length).

In the case of $\gamma < 1$, the distortion of the pipe is in the form of a damped sine wave, of wave length $2\pi/\nu$. For $\gamma \geq 1$ the distortion is critically damped. A schematic diagram of the distortion of the pipe is shown below:
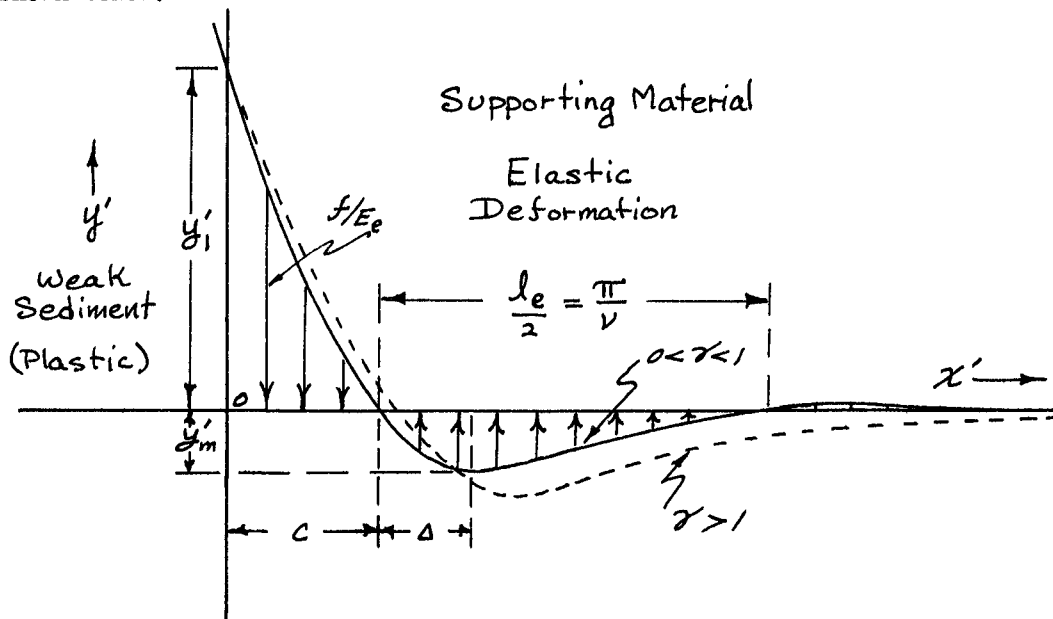
The distribution of reaction, f, is indicated by the vertical arrows.

The distortion y'_m indicated in the above diagram occurs at the distance Δ from the position of zero distortion. For the case of $\gamma < 1$:

(72)
$$\tan \nu\Delta = \frac{\nu}{\beta} = \sqrt{\frac{1-\gamma}{1+\gamma}}.$$

In terms of the wave length ℓ_e , the value of Δ lies in the range:

$$0 < \Delta < \ell_e/8 \,,$$

the upper limit corresponding to $\mathcal{T} = 0$ (no tension). The position of zero slope (corresponding to $y' = y_m'$), zero moment, and zero shear occur at distances of Δ , 2Δ , and 3Δ from the position of $y' = 0$, respectively. A special case of (67a) for the condition $N = 0$ has been investigated previously by Timoshenko (1930)*. In this case, $\beta = \nu = \alpha/\sqrt{2}$.

For all values of the tension parameter $\mathcal{T}$, i.e., for either damped sine distortion or critically damped distortion, the distortion becomes negligible in a distance of about $2\pi/\alpha$ from the end. This distance will be small if the elastic modulus of the supporting material is large. For all practical purposes, if the supporting material is at least as long as $2\pi/\alpha$, then the relations (67a,b,c) are valid.

Maximum Total Reaction and Moment, for Elastic Deformation

The total net reaction of the supporting material is denoted by F . This is defined by the relation

$$(73) \qquad F = \int_0^\infty f \, dx' .$$

In general, the value of F depends upon the arbitrary end conditions y_1' and C as well as upon N , and the characteristics of the pipe and its supporting material. The extreme value of reaction, F_{ult} , which can occur under the condition of elastic deformation of the supporting material is given by the approximate relation:

$$(74) \qquad F_{ult} \doteq \left(0.80 + 1.32 \sqrt{\gamma + 1.11} \right) \frac{f_c}{\alpha} \,, \qquad \text{for } \gamma < 10 ,$$

where f_c is the critical limit of f beyond which the deformation of the supporting material becomes plastic. The conditions for maximum reaction are that

$$y_1 = C = 0 \qquad \left(\frac{y_1}{\Delta_{in} y_c} \neq 0 \right) ,$$

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* This is an extension of the original investigation of a beam on an elastic foundation which was carried out by Winkler (1867).

and

$$|y'_m| = |y'_c| = \frac{|f_c|}{E_e} .$$

The bending moment in the pipe likewise depends upon y'_i and C in general. The extreme value which can be obtained, M_{ult}, under elastic conditions, is given by the following approximate relation:

(75)
$$M_{ult} \doteq - (3.86 + 4.20\gamma) \frac{f_c}{\alpha^2} , \quad \text{for} \quad \gamma < 10 .$$

The condition for this extreme value is that

$$y'_i = - y'_m ,$$

and

$$|y'_m| = |y'_c| .$$

It should be noted that the extreme values of F and M do <u>not</u> occur under the same conditions of C .

<u>End Conditions and Total Reaction Relationships</u>

In general the following useful relations hold for all values of γ :

(76)
$$M_1 = \frac{1}{\alpha^2} \left[(1+2\gamma) E_e y'_i + 2\beta F \right] ,$$

(77)
$$\theta_1 = - \frac{\alpha^2}{E_e} F - 2\beta y'_i ,$$

and

(78)
$$F = N\theta_1 - V_1 = \frac{1}{2} w l_o .$$

For the special case of zero tension $(\gamma = 0)$:

(76a)
$$M_1 = \frac{1}{\alpha^2} \left(E_e y'_i + \sqrt{2} \alpha F \right) , \quad .$$

(77a)
$$\theta_1 = -\frac{\alpha^2}{E_e} F - \sqrt{2} \alpha y_1' \, ,$$

and

(78a)
$$F = -V_1 = \frac{1}{2} w l_o \, .$$

Furthermore, if $y_1 = 0$, then

(79a)
$$M_1 = \frac{\sqrt{2}}{\alpha} F \, ,$$

and

(79b)
$$\theta_1 = -\frac{F}{\alpha^2 E I} = -\frac{\alpha^2}{E_e} F \, .$$

The end moment corresponding to the maximum total reaction of the supporting material (for non-plastic deformation) can be found from (79a) by substituting for F the value F_{ult} from equation (74).

The Modified Rigid End Condition for Free Sag

By eliminating the dimensionless shear factor ψ between equations (24) and (26) and employing the definition of m_1 from equation (22), the general relationship between M_1 and θ_1 at the ends of that portion of the pipe in the weak sediment zone is obtained. Another relationship between M_1 and θ_1 which must be satisfied if the pipe is supported by an elastically deformed material is given by equation (76) and (77), where y_1' is eliminated between the latter equations. Consequently, if these relations are to be satisfied simultaneously, then it follows that:

(80)
$$M_1 = \left\{ \frac{\frac{1}{2} w l_o \frac{a}{b} - F^*}{2a\beta + F^*} \right\} b \, ,$$

(81)
$$\theta_1 = \frac{M_1 + b}{a} \, ,$$

and

(82)
$$y_1 = \frac{M_1 - \frac{2\beta}{\alpha^2}F}{F^*} \; ;$$

where

(83)
$$a = \frac{N l_0}{n \tanh n/2} \; ,$$

(84)
$$b = -w l_0^2 (m_1)_{\theta_1 = 0} = -(M_1)_{\theta_1 = 0} > 0 \; ,$$

and

(85)
$$F^* = \frac{(1 + 2\gamma) E_e}{\alpha^2}$$

The maximum bending moment in the pipe occurs within the supporting material at the distance $l_m = c_m + 3\Delta$ from the position at which M_1 occurs. The value of the maximum moment M_{max} for the case of $\gamma < 1$ is given by

(85)
$$M_{max} = M_1 \frac{\sin \nu c_m}{\sin [-\nu(c_m + 2\Delta)]} e^{-\beta(c_m + 3\Delta)} \; ,$$

where c_m is given by

(86)
$$\operatorname{ctn} \nu c_m = \frac{1}{2\nu\beta} \left(\frac{M_1}{EI y_1'} - \alpha^2 \gamma \right) \; ,$$

and Δ is given by equation (72).

The equations (80) to (86) represent the generalized relationships for a "nominally rigid end condition". That is, the supporting material can be considered nominally rigid if the deformations experienced in it are below the critical limit, y_c', and consequently are small compared to the maximum sag of the pipe, which occurs in the plastically deformed weak sediment zone. The maximum bending moment which can occur in the pipe for a given sag, depends essentially upon the modulus E_e of the supporting material. For extremely large values of E_e (approaching that of concrete or steel), the deflection y_1' and the slope θ_1 become negligible and the maximum moment is essentially $(M_1)_{\theta_1 = 0}$. That is, the condition of infinite elastic modulus of the supporting material represents the truly

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rigid end condition, and is actually never attained in nature.

In order to carry out any computations for the "nominally rigid end condition", it is necessary to determine the tension factor n^2 or N itself. The value of n^2 for the extreme end conditions $\theta_1 = o$ and $m_1 = o$ can be determined. In general the value of n^2 for the "nominally rigid end condition" will lie somewhere between the values based upon the above end conditions, since the angle θ_1 will lie between zero and the value of θ_1 corresponding to $m_1 = o$. As a first approximation, the mean value of n^2 determined for the two extreme end conditions can be used. It will be noted from Figure 3 that this approximation cannot be more than 15% in error if q is greater than 10,000. For smaller values of q, the slope θ_1 can be determined from the first approximation of n^2; a second approximation for n^2 can then be obtained by interpolating between the values corresponding to $\theta_1 = o$ and $m_1 = o$, by comparing the above value of θ_1 with that corresponding to the condition $m_1 = o$. In most cases of practical interest, q will be sufficiently large, such that a second approximation of n^2 is unnecessary.

The Critical Limit of Deformation

If the deformation of the supporting material is increased to the point where significant plastic yielding of the material occurs, then the reaction becomes nearly independent of deformation. This means that the load, f, becomes essentially uniform over that portion of the pipe for which the deformation has exceeded the critical limit. As a result, the bending moment in the pipe is redistributed and the maximum value is reduced.

It was assumed in the development of the "nominally rigid end condition" that the supporting material is elastic in the sense that f is proportional in magnitude to the deformation, up to the critical limit, y_c'. In a material such as stiff clay, the load-deformation relationship* is not linear, and consequently the value of y_c' is difficult to define.

A schematic diagram of the typical load-deformation curve for clay is illustrated below:

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* As found in the laboratory as well as the settlement load relationship observed in the field.
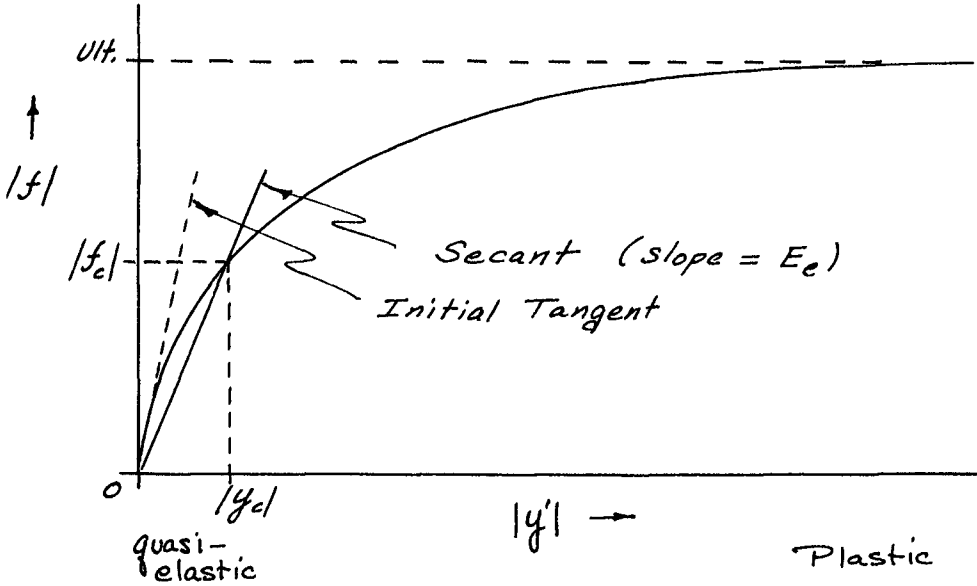
As a first approximation, the critical load, f_c , can be taken as one-half the maximum reaction of the material, and the critical deformation, y_c' , is that deformation corresponding to f_c on the load deformation curve. Consequently, the quantity $E_e = |f_c/y_c'|$ represents a secant modulus of the material; its value is roughly about one-half the value of the initial tangent modulus. The rate of increase of f with y' for deformations greater than y_c' becomes so small that for all practical purposes, the material is plastically deformed beyond this limit.

The application of equations (66) to (86) for a supporting material such as clay is admittedly an approximation. Nevertheless, the resulting bending moment is a much more reasonable estimate than that which would be obtained by considering the supporting material as rigid. Furthermore, it can be shown that if the estimate of E_e is in error by as much as 50 per cent, the resulting error in the estimated value of M_l and M_{max} is less than 12 per cent for values of E_e of the order of 10,000 lbs./ft.^{2}.

The value of M_{ult} , on the other hand, is not as accurate. The value of M_{ult} is proportional to

$$\sqrt{\frac{|f_c|}{D} \frac{|y_c'|}{D}}$$

The quantity inside the radical represents twice the <u>apparent</u> modulus of resilience of the supporting material. (Actually a material such as clay suffers a permanent deformation even for $y < y_c$, so that the true resilience is less than this.) An error of 50 per cent in the apparent modulus of resilience will lead to an error of about 25 per cent in M_{ult} .

APPLICATION: SAG OF A LIQUID FILLED TEN-INCH PIPE

<u>An Example of Free Sag</u>

Suppose that the proposed route of a ten-inch pipe line traverses a band of extremely soft sediment, which is about 500 feet wide and about 12 feet deep at the anticipated crossing, and is several miles in length transverse to the pipe. The weak sediment is sandwiched between two extensive reef bodies, consisting of uncemented oyster shells in a clay matrix.

Tests carried out in the field disclose that the weak sediment has a bearing capacity of only 70 lbs. per ft. for a coated section of the pipe*. The base pipe is 10 inches O.D. with 1/2 inch steel walls, 5/8 inch coating of Somastic, and 1 inch coating of concrete for protection. Its weight per unit length is estimated to be 95 lbs. per ft. when loaded with liquid petroleum. This takes into account the uplift due to the moisture content of the sediment, which is 43.7 lbs. per cu. ft. The reef material has a bearing capacity significantly greater than the submerged weight of the loaded pipe. Consequently differential sag of a 500 foot section of the pipe would occur under these conditions and may lead to severe stresses in the pipe.

To avoid this situation, two alternatives present themselves: (1) re-route the pipe line so as to by-pass the weak sediment zone, or (2) en-trench the pipe to a depth of about 12 feet within the reef zones so as to reduce or eliminate the anticipated vertical distortion of the pipe. Either of these alternatives might be costly, and therefore it is worth-while to determine quantitatively if the pipe could be allowed to sag with-out danger of overstressing.

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*   This bearing capacity corresponds to a maximum shearing resistance of the sediment of roughly 30 lbs. per ft.[2].

The specifications of the pipe are summarized below:

$$I.D.\ Steel = 9\ in.$$

$$O.D.\ Steel = 10\ in.$$

Overall D
(protected) $= 13\ in.$

$$A_s = 14.9\ in.^2$$

$$I = 169\ in.^4$$

$$r = 3.36\ in.$$

$$EI = 35.2 \times 10^6\ lb.ft.^2$$

$$EIr = 9.85 \times 10^6\ lb.ft.^3$$

$$R_o/r = 1.49$$

It will be assumed at first that there is no tension in the pipe prior to sag, and furthermore that there is no longitudinal slippage of the pipe in the supporting reef material. Thus the loading and length factors can be summarized as follows:

$$w = 95 - 70 = 25\ lbs./ft.$$

$$\ell_o = 500\ ft.$$

$$f_r = 0\ (\Delta = 0)$$

$$N_o = 0\ (n_o^2 = 0)$$

Therefore, using equation (20), the flexibility parameter can be computed:

$$q = \frac{25\ (500)^4}{9.85 \times 10^6} = 159,000.$$

The total net load of the sagging section of pipe is 12,500 lbs. The reef material therefore must carry 6,250 lbs. at each end of the sagging section in addition to the load exerted by the pipe line passing over the reef. If the reef material is extremely stiff then possibly this load could be carried without significant distortion of the reef material. If so the rigid end condition, $\theta_1 = 0$, might apply. However, if the reef material has a bearing capacity of only 120 lbs. per ft., then the material would be plastically deformed. The upward net load on the pipe would be about

25 lbs. per ft. in the reef material, and the moment would be released altogether at the ends of the 500 foot section. The moment distribution and deflection of the supported pipe would be similar but opposite in sign to that occurring in the unsupported pipe section. This condition represents the end condition for which $\mathcal{M}_1 = 0$.

Calculations made on the basis of the two extreme end conditions place upper and lower limits on the maximum stress that could be expected for the situation being considered. Such calculations are presented below:

Method of Determination	Case I: $\theta_1 = 0$	Case II: $\mathcal{M}_1 = 0$
Fig. 3 using $q = 159,000$	$n^2 = 920 = \sigma_t$ $n = 30.4$	$n^2 = 1,000 = \sigma_t$ $n = 31.7$
Equa. 19	$N = 130,000$ lbs.	$N = 141,000$ lbs.
Equa. 41a ($k_1 = k_2 = 1$)	$y_m = 5.33$ ft.	$y_m = 5.53$ ft.
Equa. 40a		$\theta_1 = .0398$ rad $= 2.3°$
Fig. 4	$m_1 = -.0153$	
$m_o = 1/n^2$ for $n^2 > 500$	$m_o = .00108$	$m_o = .000995$
Equa. 22	$M_1 = -95,500$ lb.-ft.	
Equa. 21	$M_o = 6,750$ lb.-ft.	$M_o = 6,220$ lb.-ft.
Equa. 52	$(\sigma_b)_{max} = 3,620$ (using m_1) $(\sigma_b + \sigma_t) = 4,540$	$(\sigma_b)_{max} = 236$ (using m_o) $(\sigma_b + \sigma_t) = 1240$
Equa. (54) using $(\sigma_b + \sigma_t)_{max}$	$(S_b + S_t)_{max} = 42,650$ psi	$(S_b + S_t)_{max} = 11,670$ psi
	occurs at ends	occurs at center

If the elastic limit of the material in tension is in the neighborhood of 30,000 psi, then the value of maximum stress for the condition $\theta_1 = 0$ indicates that the assumption of elasticity has been transcended, and the

value of stress is quantitatively in error but qualitatively is indicative of an unsafe situation. Actually, the maximum stress will lie somewhere between the limits 11,670 psi and 42,650 psi, <u>depending upon the strength characteristics of the supporting reef material</u>.

To test whether or not $S_b + S_t$ is the maximum combined stress, the value of shear stress must be computed. For the case rigid ends, $\bar{S}_s$ is a maximum, since the entire net vertical load of the pipe is carried by shear and none in tension at the ends. This end shear is 6,250 lbs, so that $\bar{S}_s = 420$ psi (from equation 47). The value of S_b is 34,000 psi (from the value of σ_b given above), consequently $4(\bar{S}_s)^2/S_b$ is only about 20 psi. From condition (A) of Table IV, this means that the combined stress $S_b + S_t$ does govern in this case as presumed, since it is very unlikely that the hoop stress is greater than 42,630 psi (this would require an internal pressure of about 4,740 psi in excess of the environmental pressure outside the pipe).

The value of sag for the case of plastic yielding of the supporting material will be greatest. For the end condition $m_1 = 0$ the center of sag will be $2y_m$ below the equilibrium position of the pipe or about 11 feet for the situation above. This is less than the depth of the weak sediment zone in this case and therefore the situation is truly one of free sag.

Effect of Initial Tension for the Rigid End Condition

A reduction in temperature of a very long pipe line below that at which the pipe was initially installed can induce an axial tensile stress in the pipe if the longitudinal restraint provided by sediments and protecting coating inhibit the contraction of the pipe.* For a 50°F change in temperature, the maximum thermal stress which can be induced in the steel is about 9,750 psi. In the 10 inch pipe examined above, this is equivalent to an axial tensile force of $N_o = 145,000$ lbs.

What affect would an initial tension of this magnitude have on the sagging section of pipe with rigid end conditions?

For the case of no end slippage as before ($\lambda = 0$, $k_2 = 1$), the value η^2 must be evaluated from the equations (28) and (29):

$$\eta^2 = \frac{\eta_o^2}{1 - \left[\dfrac{q}{f_1(n)}\right]^2} \ .$$

From equation (23), using the value of N_o above:

$$\eta_o^2 = 1,000 \ .$$

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*   For the case of initial compression see Summary and Conclusions
      and also the Appendix.   369

For the case of $m_1 = 0$, $n^2 = 1,500$ from Figure 3; however for the case of $\Theta_1 = 0$, the value is probably somewhat less than this. Using $n^2 = 1,500$ as a first approximation, the value of $f_1(n)$ can be found from the dashed curve in Figure 3 by reading the value on the q_f scale corresponding to the value n^2 above. This yields $f_1(n) = 320,000$. Using this estimate of $f_1(n)$, together with the value of q_f already computed, a second approximation of n^2 can be found as follows:

$$n^2 = \frac{1000}{\left\{ 1 - \left[\frac{159,000}{320,000} \right]^2 \right\}} = 1330 \ .$$

A third approximation following the same procedure, but using the mean value of the first and second approximations of n^2 to find $f_1(n)$, yields:

$$n^2 = 1400,$$

which is sufficiently accurate. Thus from equation (19) the final tension N is 197,000 lbs. This is an increase of only 52,000 lbs. above the initial value, and the sag corresponding to this, from equation (41a), is only 3.5 ft.

The value of maximum stress is found to be 41,100 psi, which is only slightly less than that found for the case of zero initial tension. The reason for the small difference in stress is that the increase in pure tensile stress is offset by the reduction in bending moment for the higher value of tension parameter.

Effect of End Slippage

In the previous computations it was assumed that the slippage was zero ($\Delta = 0$, or $k_2 = 1$). Strictly speaking, this requires an indefinitely large value of longitudinal restraint f_r ; however, in the problem being considered, if f_r is about 50 w or 1250 lbs. per ft., then k_2 would differ from unity by only 10 per cent. For the 10 inch nominal pipe entrenched in the supporting material, this would require a value of τ_u (equation 16) of approximately 400 lbs. per sq. ft., which is not unreasonably large.

Suppose now that f_r were only 25 lbs. per ft. ($f_r/w = 1$). This situation could obtain for a 10 inch pipe resting upon the supporting material (but not entrenched), provided that the coefficient of friction between the pipe and the underlying material were about 3/10.

The slippage coefficient is given by equation (41):

$$\Delta = \frac{25}{25} \left[\frac{30 \times 10^6 \times 14.9}{25 \times 0.28} \right]^{1/4} = 89.4 .$$

For the case of no initial tension and zero end slope, equations (28),
(29) and (30) reduce to

$$f_1(n) = \frac{159,000}{\sqrt{1 + .0112\, n^2}} ,$$

where q is taken as 159,000 as before. The value of n^2 will be lower
than in the case of no slippage; however, using $n^2 = 920$ as found
previously, the approximate value of $f_1(n)$ is 47,000 as computed from the
last equation above. This corresponds to a value of n^2 of 380 (Figure 3),
which is the first approximation of n^2. Using this value, a second
approximation, $n^2 = 510$, can be found by the same procedure. The true
value of n^2, which represents a root of the equation,

$$F(n) \equiv f_1(n) \sqrt{1 + .0112\, n^2} - 159,000 = 0 ,$$

must lie between the first and second approximation.

It is found that

$$F(n) = -51,400, \text{ for } n^2 = 380 ,$$

and

$$F(n) = 22,300, \text{ for } n^2 = 510 .$$

Hence the root as found by linear interpolation is

$$n^2 = \frac{51,400}{73,700} (510 - 380) = 470 .$$

If this is carried one step further a value of 480 is found to satisfy
$F(n) = 0$ more closely. This is of sufficient accuracy.

Using $n^2 = 480$ (or $n = 21.9$) the value of k_2 is found to be
2.52, and the maximum sag (equation 41a) is

$$y_m = 0.626 \times 2.52 \times 21.9 \times 0.28 = 9.7 \text{ ft.}$$

From Figure 4, a value of m_1 of -.021 for the case of $\Theta_1 = 0$, is
obtained. The tension and bending moments evaluated from the value of
n^2 and m_1 above are 67,600 lbs. and -131,000 lb.-ft. respectively.

Finally the stresses can be computed (equations 45 and 46):

$$S_b = 46,800 \text{ psi},$$

$$S_t = 4,500 \text{ psi},$$

and

$$S_b + S_t = 51,300 \text{ psi}.$$

The value of maximum total stress is about 20 per cent greater than in the case of no end slippage. In many cases the end slippage will have considerably less affect - the case examined here probably represents an extreme condition of slippage.

If both initial tension and end slippage exist, the problem of determining n^2 can be solved by successive approximation in a similar manner.

Modified Rigid Ends

It was stated that the maximum stress in the 10 inch pipe will lie between the limits 11,670 psi and 42,650 psi (for the case of $\triangle = 0$, $n_o^2 = 0$) depending upon the strength characteristics of the supporting material. Two different conditions of elastic deformation of the supporting material are examined here in order to give a more realistic idea of the stress.

For the case of $\triangle = 0$, $n_o^2 = 0$, the value of N was found to be 130,000 lbs. for the rigid end condition and 141,000 lbs. for released end moment. The mean value 135,000 lbs. will be presumed for the case of elastic deformation of the supporting sediment investigated below.

Suppose that the supporting material has an effective modulus (E_e) of 12,000 lbs. per ft.^{2}; this is representative of the somewhat stiffer soft clays of the Atchafalaya region. If the value of f_c for this material is not exceeded then equations of elastic deformation will apply.

The following characteristic parameters are evaluated from equations (68) through (70b):

$$\gamma = 0.10,$$

$$\alpha = .136 \text{ ft.}^{-1}, \quad \alpha^2 = .0185 \text{ ft.}^{-2},$$

$$\beta = .101 \text{ ft.}^{-1},$$

$$\nu = .0913 \text{ ft.}^{-1},$$

and

$$\ell_e = \frac{2\pi}{\nu} = 69 \text{ ft.}$$

Thus the pipe deflection in the elastically deformed supporting material is in the form of a non-critically damped sine wave (since $\gamma < 1$) with a wave length of 69 ft. If the supporting reef material is at least 69 ft. wide then the elastic theory as given here is applicable.

From equations (74) and (75) it is found that

$$F_{ult} = 16.5 \ f_c ,$$

and

$$M_{ult} = 232 \ f_c .$$

It is seen immediately from these relations that a supporting material with an f_c of only 100 lbs. per ft. could not possibly support the load or moment indicated by the condition of rigid ends. It will be found that the value of f_c must be about 20 times greater than this if the supporting material is to be free of plastic deformation.

The next step is to compute a , b , and F^* using relations (83), (84), and (85):

$$a = 2.18 \times 10^6,$$

$$b = 95,500,$$

$$F^* = 779,000 \text{ lbs.}$$

Inserting these values in equations (80), (81), and (82) yields:

$$M_1 = -49,100 \text{ lb. ft.},$$

$$\theta_1 = .0207 \text{ radians},$$

and

$$y_1' = 0.15 \text{ ft.}$$

The deflection $y_1' = 0.15$ ft. represents the extreme value of elastic distortion of the supporting material in this case, so that the lower limit of f_c is

$$f_c = 12,000 \times .15 = 1800 \text{ lbs./ft.}$$

This would require a shear strength (τ_u) of at least 830 lbs./ft.^{2} (if k_b is taken as 2.0, equation 2).

From equation (86), the condition of maximum moment requires that

$$\nu C_m = -1.95 \text{ radians},$$

and from equation (72):

$$\nu \Delta = 0.735 \text{ radians.}$$

Thus the position of maximum moment from the end of the weak zone is

$$\ell_m = c_m + 3\Delta = 2.8 \text{ ft.}$$

Using equation (85) the maximum moment can now be computed:

$$M_{max} = -54,000 \text{ lb. ft.}$$

Finally the maximum total stress can be computed

$$S_b + S_t = 28,250 \text{ psi.}$$

A similar analysis for the case of E_e = 2,000 lbs./ft.^{2} leads to a somewhat smaller total stress. The results of these computations are summarized in Table VII. In this case the pipe deflection in the supported zone is still a non-critically damped sine wave, but the wave length is about 120 ft., indicating that the distortion of the supporting material is spread out over a greater length along the pipe, leading to a re-distribution and reduction of the maximum bending moment. The strength f_c required in this case is at least 740 lbs./ft. or greater, in order that the supporting material is truly in an elastic state of deformation.

All of the computations for the example of free sag in a 10 inch pipe line of 500 foot sag length are summarized in Table VII for convenience of comparison. As mentioned previously, the condition $M_1 = 0$, or complete release of the moment at the ends of the weak zone, represents the special condition of plastic deformation of the supporting material, for which $f_c = w = 25$ lbs./ft. The problem of partially elastic and partially plastic deformation of the supporting material has not been investigated.

An Example of Restricted Sag

In the previous example the amount of total sag was found, for each of the end conditions investigated, to be less than the depth of the weak sediment zone (as measured from the equilibrium position of the pipe, which occurs at moderate horizontal distance from the weak sediment zone). The example therefore was truly one of free sag. Suppose now that the width of the weak zone is ten times that of the above example, i.e. $\ell_o = $ 5,000 feet or about 1 mile. A ten inch nominal pipe will again be considered but in this case it will be presumed that the net downward unit force on the pipe, w, is only 10 lbs. per ft. Thus if the pipe were to sag freely in the weak zone of sediment, then the supporting material at the ends would have to carry a load of 25,000 lbs. at each end of the sagging section, which is only about four times that considered in the previous example.

TABLE VII

SUMMARY OF COMPUTATIONS OF FREE SAG FOR A TEN INCH STEEL PIPE:

Net downward load in weak sediment 25 lb/ft (w)
Width of weak sediment zone 500 ft (ℓ_o)
Pipe data: $EI = 35.2 \times 10^6$ lb ft^2
$A_s = 14.9$ in^2
$D = 13.0$ in (protected)

Initial Tension End Slippage	$n_o^2 = 0$ $\ell = 0$ $(f_r > 50\,w)$				$n_o^2 = 1000$ $\ell = 0$	$n_o^2 = 0$ $f_r/w = 1$
State of Adjacent Sediment	Plastic Deformation	Elastic Deformation		Rigid	Rigid	Rigid
End Cond.	$M_1 = 0$	---	---	$\theta_1 = 0$	$\theta_1 = 0$	$\theta_1 = 0$
E_e (lbs/ft^2)	0****	2,000	12,000	∞	∞	∞
N_o (lbs)	0	0	0	0	145,000	0
N (lbs)	141,000	135,000	135,000	130,000	197,000	67,600
M_o (lb ft)	6,220	6,500	6,500	6,750	4,500	13,100
M_1 (lb ft)	0	-34,400	-49,100	-95,500	-78,700	-131,000
M_{max} (lb ft)*	±6,220	-40,700	-54,000	-95,500	-78,700	-131,000
S_t (psi)	9,450	9,050	9,050	8,650	13,200	4,500
$(S_b)_{max}$ (psi)	2,220	14,450	19,200	34,000	27,900	46,800
y_1' (ft)	5.5	0.37	0.15	0	0	0
y_m (ft)	5.5	5.4	5.4	5.3	3.5	9.7
θ_1 (rad)	.0398	.0275	.0207	0	0	0
f_c (lb/ft)	25	≥ 740	≥ 1800	∞	∞	∞
ℓ_1 (ft)***	250	28.0	12.9	0	0	0
ℓ_m (ft)*	±250	7.3'	2.8'	0	0	0
$y_1' + y_m$ (ft)	11.0	5.8	5.6	5.3	3.5	9.7
$(S_t + S_b)_m$ (psi)	11,670	23,500	28,250	42,650**	41,100**	51,300**

* Maximum moment occurs at distance ℓ_m from end of weak zone (or $\ell_o/2$ + ℓ_m from center of sag)

** Stress which would exist if the elastic limit of the material were great enough

*** Equilibrium level of pipe reached at distance ℓ_1 from end of weak zone

**** Limiting case of $/f/ = /w/$

However the flexibility parameter, q , (using the same values of E, I, and r as before) is 6.34×10^8 or 4,000 times greater than before. For such an extreme flexibility, the pipe acts as a flexible cable and the asymptotic equation (36) can be used to compute the tension parameter. In fact for $\Delta = 0$, $n_o^2 = 0$:

$$n = \left(\frac{q}{4.9} \right)^{1/3} = 505;$$

and according to equation (41a) the maximum free sag, ψ_m, would be about 89 feet! The tensile stress due to elongation alone, according to equation (37a) and/or (46), would be 24,200 psi, and if the end conditions are considered rigid then a maximum total stress of over 100,000 psi could be developed (see Table V).

It is inconceivable that an extremely weak sediment zone of a depth comparable to the computed free sag above could exist. Consequently the pipe will actually sag until it rests upon the firmer sediments at the base of the weak zone, and the full elongation of the pipe, expected in the case of free sag, will not be realized.

Suppose that the average depth of the weak zone h_o (Figure 2A) is 10 feet. If the adjacent supporting material is of sufficient strength, then the difference $h_o - h$ will be small, and hence h can be approximated by h_o . Consequently, under this condition the relative depth h/r will be 35.7 and according to equation (61) the tension parameter for the length of pipe ℓ_o' (Figure 2B) will lie in the range:

$$255 < (n')^2 < 893,$$

provided that no initial tension existed preceding the sag of the pipe into the weak zone.

For the condition $q = 6.34 \times 10^8$ ($w = 10$ lbs./ft. and $\ell_o = 5,000$ ft.), equation (58) reduces to

$$(58a) \qquad \left[\frac{(n')^2}{255} - 1 \right]^4 - 1.57 \times 10^{-5} \frac{f_2(n')}{n'} = 0$$

Using the range of $(n')^2$ stated above as a guide, the appropriate root of this equation can be ascertained by successive approximation, or by graphical procedure. Since in this case the unrealized free sag is much greater than the depth of the weak sediment zone, it is expected that the ratio ℓ_o'/ℓ_o will be quite small. Consequently, the value of $(n')^2$ will probably lie closer to the lower limit 255 than to the upper limit. Using this as a starting point, the value of $f_2(n')$ as determined from Figure 3 is 20,500 (full curve labled $n_o^2 = 0$; $f_2(n')$ read from the q scale).

By evaluating the approximate value of the ratio $f_2(n')/n'$ (using $n' = 16$), a first approximation of $(n')^2$ can then be determined explicitly from (58a). This yields $(n')^2 = 350$.

A second approximation evaluated in the same way but using $(n')^2 = 350$ to obtain $f_2(n')/n'$ yields $(n')^2 = 360$. The convergence to the final value is quite rapid in this case (a third approximation yields the same result to within one per cent accuracy).

Using $(n')^2 = 360$, the length ratio ℓ_o'/ℓ_o according to equation (59) is .082, or $\ell_o' = 410$ feet. The tension accordingly is 75,500 lbs. (equation 60), which corresponds to a tensile stress, S_t, of about 5,100 psi. The above tension actually applies to the case where the moments at both ends of the section ℓ_o' are zero. However it will be noted that for the value of q' corresponding to $(n')^2 = 360$, the values of $(n')^2$ for the two extreme end conditions differ by only 20 per cent (Figure 3). Consequently it is presumed that the tension found above is approximately valid for different end conditions which might be considered.

If it is supposed now that the supporting material adjacent to the region of sag is quasi-rigid so that the pipe is essentially horizontal, then the moment factor at this end is given approximately by equation (63). Using $(n')^2 = 360$ and q' as found from equation (64) ($\ell_o'/\ell_o = .082$), the calculated moment factor is about -.047, and the corresponding bending moment is about -79,000 lb.-ft. (equation 62b). This corresponds to a maximum bending stress of 28,000 psi and consequently the maximum combined stress in the pipe is approximately

$$(S_b + S_t) = 33,000 \text{ psi.}$$

It will be noted that this applies for the case of $n_o^2 = 0$, $\Delta = 0$ and $\Theta_1 = 0$.

The effect of end slippage and/or elastic deformation of the supporting material could be carried out in a manner similar to that already presented in the problem of free sag. However, due to the complexity of the restricted sag problem and the nature of the approximations already made in this application, it would appear that such refinements are not justifiable, unless the asymmetry of the section ℓ_o' is likewise taken into account. This would require re-examination of the basic equation (4) and its solution (8); for in this case a finite shear V_o would exist at the center of the sagging pipe section.

On examining the question further, a better approximation might be had by considering the section ℓ_o' as one-half of a symmetrical, freely sagging section formed by deleting the section of pipe which is fully supported at the base of the weak sediment zone. The difficulty in applying

the free sag theory in this case is that there exists a finite shear
and zero moment at the point of tangency, while the free sag theory presumes
no shear at the center of sag and yields a finite bending moment at that
point.

SAG OF A TWENTY-INCH PIPE

The specifications of a 20.5 inch O.D. steel pipe (thickness 3/4 inch)
are given in Table III. The value of the product EI_r for this pipe is
275×10^6 lb. ft.^{2}. If this pipe has a 5/8 inch coating of protective
Somastic and a 1 inch coating of reinforced concrete, the total weight in
air (gas filled) would be about 269 lbs. per ft. The over-all diameter
of the protected pipe would be 23.75 inches.

It will be presumed that this pipe sags into a weak material identical
to that discussed in the case of the ten inch nominal pipe. This implies
that if the bearing capacity of the sediment is proportional to the diameter
of the pipe, as equation (2) indicates, then

$$P_b \doteq P_m = \frac{23.75}{13.00} \times 70 = 128 \text{ lbs./ft.}$$

Furthermore the bouyancy due to moisture content of the sediment (for an
entrenched pipe) would be 134 lbs./ft. ($B = 43.7$ lbs./cu. ft. as before).
Thus the net vertical force w per unit length would be only 7 lbs. per ft.
for a gas filled pipe. This is of the same order of magnitude as the
errors involved in the estimate of P_u and/or B . Consequently one should
investigate the influence of the possible error in w on the resulting
stress, in order to see if the computation leads to an unqualified decision
regarding the vertical stability of the pipe.

If the pipe is to transport liquid having a specific gravity approach-
ing that of water, then the value of w would be about 130 lbs. per ft.
For a given span length of sag, it is evident that there will be considerable
difference in the maximum pipe stresses induced by these two limiting
conditions of loading. Computations based on these two cases, for a span
length of 500 feet (for comparison with the example of free sag of a ten
inch pipe), are summarized below.

Free Sag of a Gas Filled Pipe

In this case $w = 7$ lbs./ft. and $\ell_o = 500$ ft., which yields a value
of q equal to 1590. The corresponding value of n^2 from Figure 3 is 19.2
for the conditions: $\Delta = 0$, $n_o{}^2 = 0$, and $\Theta_1 = 0$. The maximum sag,
y_m , under these conditions is only 1.6 feet. Furthermore from Figure 4,
$m_1 = -.065$, and the computed stresses are:

$$S_b = 6,200 \text{ psi,}$$

$$S_t = 800 \text{ psi,}$$

and

$$S_b + S_t = 7,000 \text{ psi.}$$

The maximum combined stress of 7,000 psi would occur if the supporting material at the ends were rigid; otherwise the maximum stress would be less than this. However any slippage at the ends would tend to increase the stress (as found in the case of the ten inch pipe).

If the probable error in w is taken as $\pm$ 7 lbs./ft., it is evident that the value of expected stress ranges between the limits of zero and some upper limit. Figure 6 can be used to facilitate the computation of this upper limit. By doubling the value of w , and hence q the dimensionless combined stress parameter (for $\theta_1 = 0$) is increased from about 170 to 290. The combined stress itself will be increased in the same proportion, consequently the upper limit is about 12,000 psi. This indicates that an error of even $\pm$ 100 per cent in the estimated w , in this case, would not invalidate a qualitative decision with regard to the safety of this pipe line. Such clear cut results, however, appear to be the exception rather than the rule, and the decision regarding safety usually must be qualified by a statement regarding the degree of risk involved, unless positive steps are taken to avoid, partially or completely, the conditions of sag which are anticipated.

Free Sag of a Liquid Filled Pipe

In this case w = 130 lbs./ft. and ℓ_o = 500 ft., yielding a value of q of 29,800. The value of n^2 is found to be 270 for the conditions $\not{\!A} = 0$, $n_o^2 = 0$, and $\theta_1 = 0$. Corresponding to this, the maximum sag is 5.98 feet, which is very nearly the same as for the water filled ten inch pipe.

The resulting stresses induced by sag are:

$$S_b = 47,700 \text{ psi,}$$

$$S_t = 11,000 \text{ psi,}$$

and

$$S_b + S_t = 58,700 \text{ psi.}$$

An error of $\pm$ 7 lbs./ft. in w , in this case, represents a relative error of only $\pm$ 5.4 per cent; and the corresponding error in the combined stress is only about 2/3 of this or 3.6 per cent. Under these conditions the maximum combined stress above is presumably accurate to within $\pm$ 2,100 psi. This would not influence the qualitative interpretation of this result, nor

would the effect of an error in w of as much as $\pm$ 20 lbs./ft.

The magnitudes of stresses involved in these two examples bear out, quantitatively, the extremely different situations which might be realized for a gas filled pipe on one hand and a liquid filled pipe on the other.

SUMMARY AND CONCLUSIONS

In the planning and installation of a marine pipe line, the question of vertical stability can be important enough to warrant serious consideration especially if the pipe is to be laid upon or entrenched within a marine sediment of the type existing on the Gulf shelf. The important points pertaining to this question are set forth below.

1. A knowledge of the structure and strength of the sediments along the path of a proposed pipe line is essential if definite conclusions are to be reached regarding the question of vertical stability of the pipe. Regions in which the sediment bearing capacity is not sufficient to support the submerged weight of a pipe line can be disclosed only by appropriate investigation in the field.

2. The difference in strength characteristics between extremely weak zones and adjacent supporting material, together with the horizontal and vertical dimensions of the zone of weak sediment along the path of the pipe line, are the prime environmental factors in the determination of differential sinking of a pipe and consequently in the determination of the maximum stresses associated with such sag.

3. In general the determination of the stresses induced by sag depends upon the following basic parameters:

w determined by pipe weight and by the weak sediment characteristics

l_o, h_o dimensions of the weak zone

E, I, A_s, r pipe specifications (actually $r = \sqrt{I/A_s}$)

N_o, f_r, f_c, E_e initial and end conditions (rigidity factors of supporting material are f_c and E_e)

For the case of free, symmetrical sag, the following functional relations hold

$$n^2 \equiv \frac{N \ell_o^2}{EI} = F_1(q, \Delta, n_o, \text{end rigidity}),$$

$$m \equiv \frac{M}{w \ell_o^2} = F_2(n),$$

$$\theta_1 \frac{\ell_o}{r} \doteq 2.24 \frac{q}{f(n)} n,$$

$$\frac{y_m}{r} = 0.626 \frac{q}{f(n)} n,$$

and

$$S_b + S_t = E \left(\frac{r}{\ell_o}\right)^2 \left[\left(\frac{R_o}{r}\right) m q + n^2\right],$$

where

$$q \equiv \frac{w \ell_o^4}{EIr}, \quad \text{Flexibility parameter;}$$

$$\Delta \equiv \frac{w}{f_r}\left(\frac{EI}{w r^3}\right)^{1/4}, \quad \text{End slippage parameter;}$$

$$n_o^2 \equiv \frac{N_o \ell_o^2}{EI}, \quad \text{Initial tension parameter.}$$

If the end slippage is zero then

$$\theta_1 \doteq 2.24 \sqrt{\frac{(N - N_o)}{E A_s}},$$

and

$$y_m \doteq 0.626 \sqrt{\frac{(N - N_o)}{E A_s}} \ell_o.$$

If in addition $N_o = 0$, then

$$f(n) = q$$

This implicit equation for n in terms of q is represented graphically in Figure 3, and in tabulated form in Table VIII, for two different end conditions. The relationship $m = F_2(n)$ is shown graphically in Figure 4.

4. The dependence of maximum combined stress on flexibility is shown conveniently by the non-dimensional plots of Figure 6. The dimensionless stress factor,

$$\sigma_b + \sigma_t \equiv \frac{S_b + S_t}{E} \cdot \left(\frac{\ell_o}{r}\right)^2,$$

is presented graphically for the two limiting end conditions $\theta_1 = 0$ and $M_1 = 0$ and for the case of no initial tension and no end slippage. The stress parameter increases in direct proportion to q for very small values of q, corresponding to the theory of simple flexure ($S_t = 0$ or $N = 0$). For very large values of q, the stress factor becomes proportional to $q^{2/3}$, corresponding to a flexible cable. The proportionality coefficient in either limiting case depends upon the end condition. It is important to note that the condition of no end moment leads to the greater stress for low values of q; but the situation is reversed for large values of q (> 280) since the condition of rigid ends in this case leads to greater stress. In summary, for small flexibility:

$$\left(S_b + S_t\right)_m = m\, w\, \ell_o^2\, \frac{R_o}{I} \, , \left\{ \begin{array}{l} |m| = \frac{1}{12} \, , \text{ for } \theta_1 = 0 \\ |m| = \frac{1}{8} \, , \text{ for } m_1 = 0 \end{array} \right\} \, ;$$

while for large values of q :

$$\left(S_b + S_t\right)_m = k\left[E\left(\frac{w\,\ell_o}{A_s}\right)^2\right]^{1/3} , \left\{ \begin{array}{l} k = 1.548, \text{ for } \theta_1 = 0 \\ k = 0.347, \text{ for } m_1 = 0 \end{array} \right\} \, .$$

5. In general the sagging pipe section will carry its load by the vertical component of tension as well as by cross-sectional shear. For small flexibility, however, the tension is negligible and the load is carried entirely in shear. As the flexibility is increased the tension becomes more and more important, and in the limiting case of extremely large flexibility ($> 10^6$), the pipe carries its entire load by tension, except in the case of rigid end conditions, where the shear is still important but only immediately adjacent to the ends of the sagging section.

6. The situation of free sag is subject to the condition that $y_m < h$. If this is not the case then the pipe can receive additional support at the base of the weak zone. The problem of such restricted sag can be solved, in the first approximation, by considering the unsupported portion of the pipe at the ends of the sagging section as a situation of symmetrical free sag. The length of this equivalent free sag section is unknown in this case, but can be determined if h is known.

7. Two clear cut end conditions have been examined in some detail:

(I) No end slope, representing the condition for which the supporting material is nominally rigid and free of any significant deformation, implying that

$$\left| f_c / w \right| \text{ is very large } (> 1000),$$

and

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$$\left| E_e / E \right| \quad \text{is of the order of unity}$$

(II) No end moment, representing the condition for which
the supporting material is plastically deformed
at the ends by the amount $h_o - h = y_m$, which
requires that

$$\left| f_c / w \right| = 1,$$

and

$$E_e = 0.$$

The intermediate case of quasi-rigid conditions, representing that of finite
elastic distortion of the supporting material, has also been examined.
Values of f_c and E_e intermediate between the values above can be taken
into account in a correction factor to be applied to the bending moment for
a rigid end condition (equation 80). The effect of distortion of the
supporting material at the ends of the sagging section of pipe is always
to reduce the maximum bending moment induced in the material. In the
examples worked out for the 10 inch nominal pipe, the limiting case of plastic
distortion which was examined gave a value of maximum stress which is about
25 per cent of that which would be realized for rigid ends. The effect
of elastic distortion of the supporting material is less pronounced.

8. The effect of end slippage is to exaggerate the severity of the
maximum combined stress. The tension in this case is reduced but this, in
turn, is associated with a greater proportionate increase in bending moment.

9. The effect of initial tension is to decrease the severity of the
maximum combined stress, since the bending moment is decreased in greater
proportion than the increase in tension.

10. The effect of initial compression has not been examined in the
examples, but would lead to an increase in the severity of the maximum
combined stress. Although the equations for free sag given here are restricted
to the case of positive N (real values of n), the same restriction is
not imposed upon N_o . That is, negative values of N_o (or initial
compression) can be taken into account provided that this compression is not
too great (see Appendix). Such compression could be induced by a sudden
temperature increase in the pipe line, if the latter is inhibited from
expanding in the longitudinal direction. In the event of sag, this
longitudinal compression is released and in turn a tension will be developed,
provided that the loading is great enough.

A ten inch nominal pipe under the same conditions of sag as presumed for
the data of column 6 of Table VII except that n_o^2 is -1000 (corresponding
to 145,000 lbs. initial compression) would yield a maximum combined stress
of 46,200 psi. The maximum sag in this case would be 7.2 feet, which is

about twice that found for the case of 145,000 lbs. initial tension. However the combined stress is only about 10 per cent greater.

11. The examples of sag worked out for the liquid filled 10 inch and 20 inch nominal pipes serve to demonstrate that considerable stresses can be induced by sag of the pipe into a zone of extremely soft sediment (such as found in the Atchafalaya Bay region) if such a zone is contained between material which is considerably stiffer.

REFERENCES

Leipper, D. F., W. C. Thompson, P. L. Horrer, R. O. Reid, C. E. ZoBell, and T. R. Folsom (1951). Oceanographic Analysis of Marine Pipe Line Problems (Atchafalaya Bay, Louisiana and Adjacent Continental Shelf Area); Final Report on Research conducted by the Agricultural and Mechanical College of Texas for the United Gas Pipe Line Company through the Texas A & M Research Foundation. June 30, 1951.

Petroleum Engineer, The (1951). Marine studies for off-shore pipe lines; The Petroleum Engineer, March, 1951. (Reprinted from the United Gas Log, vol. 12, February, 1951.)

Reed, Paul (1951). United lays first off-shore big inch line; The Oil and Gas Journal, pp. 272-273, October, 1951.

Reid, R. O. (1951). Oceanographic considerations in marine pipe line construction; Gas Age, vol. 107, no. 9, 5 pp., April 26, 1951.

Terzaghi, Karl (1943). Theoretical Soil Mechanics; John Wiley and Sons, Inc., New York, 1943.

Terzaghi, Karl and R. B. Peck (1948). Soil Mechanics in Engineering Practice; John Wiley and Sons, Inc., New York, 1948.

Timoshenko, S. (1930). Strength of Materials; Part II: Advanced Theory and Problems; D. Van Nostrand Co., Inc., New York, 1930.

Winkler, E. (1867). Die Lehre v. d. Elastizität u. Festigkeit, Prag, p. 182, 1867.

SOME OCEANOGRAPHIC AND ENGINEERING CONSIDERATIONS IN MARINE PI LINE CONSTRUCTION

APPENDIX I

Figure 3 is not in a form that is adequate for actual use. In order to make it convenient for the plotting of the functions $f_1(n)$ and $f_2(n)$ on a detailed, large scale logarithmic grid, the computed values of these functions for the selected values of n used in the construction of Figure 3 are given in Table VIII. These values, obtained from equations (32) and (33) have been evaluated by computing machine for the lower range of n and by slide rule for the values of n greater than 4. The values are considered accurate to within $\pm$ 0.5 per cent.

In construction of the logarithmic plots of the functions $f_1(n)$ and $f_2(n)$ it is convenient to construct the straight line asymptotes given by equations (35a,b) and (36) as a guide.

Isolines for both positive and negative values of n_o^2 can be plotted on the n^2 versus q diagram by making use of equation (28) for the case of $\beta = 0$:

$$ q = \sqrt{1 - n_o^2/n^2} \ f(n) $$

The isolines for the case of negative n_o^2 will lie to the right of the curve for $n_o^2 = 0$, i.e. the curve $q = f(n)$, and will be asymptotic to the latter curve for large values of q. For small values of n^2 the isoline for negative n_o^2 will approach a constant value of q, having the value:

$$ 246 \ |n_o^2| \ , \text{ for } \theta_1 = 0, $$

or

$$ 4.90 \ |n_o^2| \ , \text{ for } m_1 = 0. $$

Thus for a given value of q it is seen that there is an upper limit of initial compression beyond which the present theory fails to give a solution, because n^2 itself becomes negative.

If n_o^2 is just at the critical value for a given q then the tension parameter is zero, and the stress in the pipe is purely flexural and is given by the simple bending theory. This stress is always greater than that applying to the case of zero initial compression, as can be seen from Figure 6.

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TABLE VIII

COMPUTED VALUES OF THE FUNCTIONS $f_1(n)$ AND $f_2(n)$
FOR SELECTED VALUES OF n

n	n^2	$f_1(n)$ $(\theta_1 = 0)$	$f_2(n)$ $(m_1 = 0)$
.01	.0001	2.46	.49
.1	.01	24.6	4.8
.3	.09	73.9	14.5
1.0	1.00	252	53.8
1.3	1.69	335	74.2
2	4	540	137
3	9	903	275
4	16	1,374	511
5	25	1,990	858
6	36	2,780	1,380
7	49	3,790	2,060
8	64	5,040	2,905
10	100	8,410	5,410
15	225	23,200	17,320
20	400	50,200	40,200
30	900	155,000	133,800
50	2,500	672,000	615,000
100	10,000	5,120,000	4,900,000
1,000	1,000,000	4,900,000,000	4,900,000,000

In the case where the initial tension is greater than the critical limit, then N is negative and the equations for pipe distortion are modified. The distortion of the pipe given by equation (8) takes on the form

$$y = \frac{1}{N^*}\left\{\left(M_0 + w\,\lambda^{*2}\right)\left(1 - \cos \frac{x}{\lambda^*}\right) + \frac{1}{2}w\,x^2\right\}$$

where

$$N^* = -N \quad \text{and} \quad \lambda^* = \sqrt{\frac{EI}{N^*}}$$

In the special case where $w = 0$, it can be shown that a critical compression,

$$N_c^* = EI\left(\frac{2\pi}{l_0}\right)^2$$

exists. If the initial compression is less than this then the pipe is absolutely stable with regard to transverse deflection. If the initial compression is greater than N_c^* then the pipe will deflect transversely, but in so doing, the initial compression is partially relieved and a definite equilibrium with an associated maximum bending moment is developed. This situation would represent a condition of quasi-buckling since there is but one form which can be assumed by the pipe for a given value of N_0^*. The transverse deflection is not severe unless N_0^* is considerably greater than N_c^*.

APPENDIX II

LIST OF SYMBOLS

Symbol		Dimensions
a	A characteristic quantity in the modified rigid end equation	lb. ft.
A_s	Cross-sectional area of the steel pipe	sq. ft.
b	Magnitude of maximum bending moment for the case of rigid ends	lb. ft.
B	Bouyant force acting on pipe per unit displaced volume in the sediment .	lbs./cu.ft.
c	Value of x' at which $y' = 0$ for elastic deformation of supporting material	ft.
D	Over-all diameter of the protected pipe	ft.
D_i	Inside diameter of the steel pipe	ft.
D_o	Outside diameter of the steel pipe	ft.
E	Elastic modulus of the steel pipe	lbs./sq.ft.
E_e	Effective modulus of the supporting material at the ends of the sagging section	lbs./sq.ft.
f	The net upward force per unit length at position x' exerted on the pipe by the supporting material	lbs./ft.
f_c	Limit of f beyond which the deformation of the supporting material becomes plastic	lbs./ft.
f_r	The longitudinal restraint per unit length of pipe exerted by the supporting material	lbs./ft.
$f(n)$	A function of n and the end condition	dimensionless
$f_1(n)$	A function of n only, for the condition $\theta_1 = 0$	dimensionless
$f_2(n)$	A function of n only, for the condition $m_1 = 0$	dimensionless
F	Total net vertical reaction exerted by the supporting material	lbs.

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Symbol		Dimensions
F_{ult}	Extreme value of F which can be sustained by an elastically distorted sediment	lbs.
F^*	A characteristic quantity used in the modified rigid end equation	lbs.
g	Acceleration of gravity, 32.2	ft./sec.^{2}
h	Vertical distance between the base of the weak zone and the vertical position of the pipe at $x = \ell_o/2$ (Figure 2)	ft.
h_o	Depth of the weak zone below equilibrium position of the pipe (Figure 2)	ft.
I	Cross-sectional moment of inertia of the pipe, taken about the neutral surface	ft.^{4}
k_b	Coefficient of proportionality between T_b and $\tau_u D$	dimensionless
k_1	Coefficient dependent upon n_o^2/n^2	dimensionless
k_2	Coefficient dependent upon $\Delta, q, n^2 - n_o^2$	dimensionless
k_3	Coefficient having the approximate value 2.24	dimensionless
k_4	Coefficient having the approximate value 0.626	dimensionless
ℓ	Length of pipe between points ($1'$) and (1) after sag occurs (Figure 1)	ft.
ℓ_e	Wave length of the non-critically damped elastic deformation curve	ft.
ℓ_i	Length of pipe between points ($1'$) and (1) before sag occurs (Figure 1)	ft.
ℓ_m	Distance from end of weak zone at which maximum moment is attained	ft.
ℓ_o	Length of weak sediment zone measured along the pipe line	ft.
ℓ_o'	Effective length of free sag in the restricted sag problem	ft.

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Symbol		Dimensions
$(\ell_o)_c$	Upper limit of ℓ_o for a given w and given pipe specifications, beyond which the simple bending theory is not valid	ft.
m	Bending moment factor at position x	dimensionless
m_o	Bending moment factor at $x = 0$	dimensionless
m_1	Bending moment factor at $x = \ell_o/2$	dimensionless
m'	Bending moment factor for length ℓ_o'	dimensionless
M	Bending moment at position x	lb. ft.
M_o	Bending moment at position $x = 0$	lb. ft.
M_1	Bending moment at position $x = \ell_o/2$	lb. ft.
M_{max}	Maximum bending moment developed in the pipe for given end conditions	lb. ft.
M_{ult}	Extreme bending moment which can be developed in a pipe in an elastically deformed sediment	lb. ft.
n^2	Tension factor	dimensionless
n_o^2	Initial tension factor	dimensionless
n_1	Characteristic wave number, elastic theory	ft.$^{-1}$
n_2	Characteristic wave number, elastic theory	ft.$^{-1}$
$(n')^2$	Tension factor for the length	dimensionless
$(n_o')^2$	Initial tension factor for the length	dimensionless
N	Axial tensile force in the pipe	lbs.
N_o	Initial axial tension prior to sag	lbs.
p	End shear factor	dimensionless
P_b	Ultimate load bearing capacity of the sediment per unit length of pipe	lbs./ft.
P_m	Maximum vertical restraint exerted on pipe by the plastically deformed weak sediment	lbs./ft.

Symbol		Dimensions
q	Flexibility parameter	dimensionless
q_c	Critical value of q beyond which the simple theory of bending is not valid	dimensionless
Q	Moisture content of sediment expressed as per cent of dry weight	dimensionless
r	Radius of gyration of cross-section of steel pipe taken about the neutral surface	ft.
R_i	Inside radius of steel pipe	ft.
R_o	Outside radius of steel pipe	ft.
s	End slippage coefficient	dimensionless
S_b	Flexural stress in the steel pipe farthest from the neutral surface of bending	lbs./sq.ft.
S_h	Hoop stress in the pipe due to internal pressure	lbs./sq.ft.
$\bar{S}_s$	Shear stress associated with V	lbs./sq.ft.
S_t	Tensile stress associated with N	lbs./sq.ft.
$S_{s,m}$	Maximum combined shear stress	lbs./sq.ft.
$S_{t,m}$	Maximum combined normal stress	lbs./sq.ft.
V	Cross-sectional shear force at x	lbs.
V_1	Cross-sectional shear at $x = l_o/2$	lbs.
w	Net downward force per unit length exerted on pipe in the weak sediment zone	lbs./ft.
w_p	Unit weight of pipe in air (including weight of contained fluid)	lbs./ft.
x	Horizontal distance measured from the center of sag along the pipe line	ft.
x'	Horizontal distance measured from the end of the weak zone into the supporting material	ft.

Symbol		Dimensions
y	Vertical distance measured upwards from the position of maximum sag	ft.
y'	Vertical distance measured upwards from the equilibrium level of the pipe	ft.
y_c'	Critical value of vertical deflection beyond which the supporting material becomes plastically deformed	ft.
y_m	Maximum sag of the pipe in the weak zone, for the length l_o	ft.
y_m'	Value of y' at which $\theta = 0$	ft.
y_1'	Vertical deformation of the pipe at the position $x' = 0$, or $x = l_o/2$	ft.
α	A characteristic wave number, elastic wave theory	ft.$^{-1}$
β	A characteristic wave number, elastic wave theory	ft.$^{-1}$
γ	A tension parameter in the elastic theory	dimensionless
Δ	Horizontal distance between the position for which $y' = 0$ and $\theta = 0$ in the elastic wave	ft.
θ	Slope of the pipe at position x	dimensionless
θ_1	Slope of the pipe at $x = l_o/2$	dimensionless
θ_1'	Effective slope of the pipe at ends of the length l_o'.	dimensionless
λ	A characteristic length in the free sag theory	ft.
ν	A characteristic wave number, elastic wave theory	ft.$^{-1}$
ν'	A characteristic wave number, elastic wave theory	ft.$^{-1}$
π	3.1416...	dimensionless

Symbol		Dimensions
ρ_s	Wet density of the sediment	slugs/cu.ft.
σ_b	Flexural stress factor	dimensionless
σ_t	Tensile stress factor	dimensionless
τ_u	Ultimate shear strength of the weak sediment	lbs./sq.ft.
τ_u'	Ultimate shear strength of the supporting material	lbs./sq.ft.